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Paper 1529 February, 1958 PO 1

Journal of the
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THE HAAS HYDROELECTRIC POWER PROJECT

J. Barry Cooke,* M. ASCE
(Proc. Paper 1529)

SYNOPSIS

Engineering considerations, design criteria and construction data for a high head hydroelectric project in granite country are presented in this paper. Of special interest is the underground power house which is underground for economic reasons. Excavation of the underground plant has been completed, and methods and progress data are given.

Major features in addition to the underground plant are: a 290-ft. high concrete face rockfill dam, an underground free discharge valve installation for the dam outlet works, an underground power tunnel intake valve, a 6-1/4 mile long high head unlined pressure tunnel, a storage sand trap, a combined gallery—simple—differential surge chamber, two six-jet vertical shaft impulse turbines, and a high head surface and underground penstock.

The Haas Project

The Haas Power Project is one of three Pacific Gas and Electric Company projects on the North Fork of the Kings River in California.[2,4] A flow of 760 cfs and static head of 2448 feet will develop 128,000 kw in a two-unit underground plant. This will be the first large underground hydroelectric plant in the United States[8] and is scheduled for initial operation in December 1958. The plant is underground for economic reasons.

A general characteristic of the Haas Project is the extent of underground construction and the use of granite rock from the massive granite formation of the Sierras. There are few cleavage planes, some zones of decomposed granite and occasional soft seams. In addition to taking advantage of the

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granite excavations, granite is used in the rockfill dam and crushed granite from the power house excavation is used for concrete aggregate.

The Kings Development, Fig. 1, develops all of the 5650 ft. of static head between maximum Wishon water surface and the Corps of Engineers' Pine Flat Reservoir. The hydrology and a general description of the overall development is presented in the References.(4) In planning the Haas portion of the development both ends were fixed. Wishon dam was fixed at the only storage dam and reservoir site and the afterbay location was fixed by the Balch diversion dam. Five alternate schemes to develop the 2448-ft. head were studied. Two schemes used a single powerhouse and three used two powerhouses. The annual cost of power for the adopted alignment with one plant was a minimum with a conventional surface penstock and powerhouse layout. The final use of the underground arrangement improved the economics.

Dam and Conduit Design

General

The use of a rockfill dam and an unlined tunnel is similar to the Company's Bear River Project, completed in 1952.(1) Different conditions and detailed review of experience have resulted in simplified details and different appurtenant structures for the dam and tunnel at Haas. The conduit includes tunnel, rock trap, surge chamber and penstock.

Concrete Face Rockfill Dam

Courtright and Wishon dams, Fig. 1, both supply stored water to the three downstream power projects. This paper will specifically discuss only Wishon, Fig. 2, since it is the diversion to Haas Project. The design and specifications are essentially identical for both dams.

There was only one site for Wishon Dam. An exposed granite ridge projected into the canyon at the downstream end of a meadow providing both a dam and reservoir site. Preliminary designs and estimates were made for three types of rockfill dam and a concrete gravity section. The three rockfill designs were thin central core, thin sloping core, and concrete face, and their estimated costs were about the same - assuming core material was available within about five miles. Explorations failed to locate sufficient suitable core material in the meadows or in decomposed granite zones and a core rockfill dam was not feasible. A concrete gravity type dam was eliminated since its cost would have been nearly twice as great as that of the concrete face rockfill dam which was adopted.

The section of the dam, Fig. 2, is different from the Company's Salt Springs and Lower Bear River designs. All the changes result in a lower cost dam and in most part are due to the very successful experience with the Lower Bear River Dams.(1) The vertical component of settlement at the maximum section of Dam No. 1 (245 ft.) was 1.12 ft. at the end of the first year and increased only to 1.30 ft. by the end of the fourth year. Maximum leakage in the four years of service has been 3.0 cfs with none from Dam No. 2 and most from the knoll between the two dams. There have been no face cracks in Dam No. 2 and only two of limited extent in the face of Dam No. 1. These were filled with Indaco, a rubberized asphalt joint filler. These favorable measures of performance, along with observations during

PHYSICAL DATA	HAAS	BALCH (Enlarged) Existing Units	BALCH (Enlarged) Additional Units	KINGS RIVER
HEAD				
Static - ft	1,444	2,379	2,389	793
FLOW				
Peak - c.f.s.	760	200	560	760
OUTPUT				
Peak - kw	178,000	32,000	94,000	47,000
Energy - mill. kwh	388	477		153
DIVERSION AND/OR STORAGE DAM	COURTRIGHT	WISHON		
Type	Rock Fill	Rock Fill	Arch	Arch
* Height - ft	290	250	118	118
Effective Capacity - A.F.	492,000	128,000	970	220
CONDUIT				
Tunnel - size - ft	13 ⌀		12 x 12	13 ⌀
length - ft	32,800		19,300	18,400
Siphon - dia. - in				114 to 98
length - ft				970
Tailrace Tunnel - size - ft	15 x 17'-6" ⌀			
length - ft	1900			
PENSTOCK				
Pipe - No. & Dia. - in	1 @ 96 to 78	1 @ 60 to 48	1 @ 96 to 68	1 @ 96 to 80
length - ft	4560	4980	5030	1770
INSTALLATION				
Number of Units	2	1	2	1
Total Prime Mover - H.P.	184,500	40,000	134,000	50,000
Total Generator - Kva	150,000	33,000	108,000	49,020

* Above streambed at axis

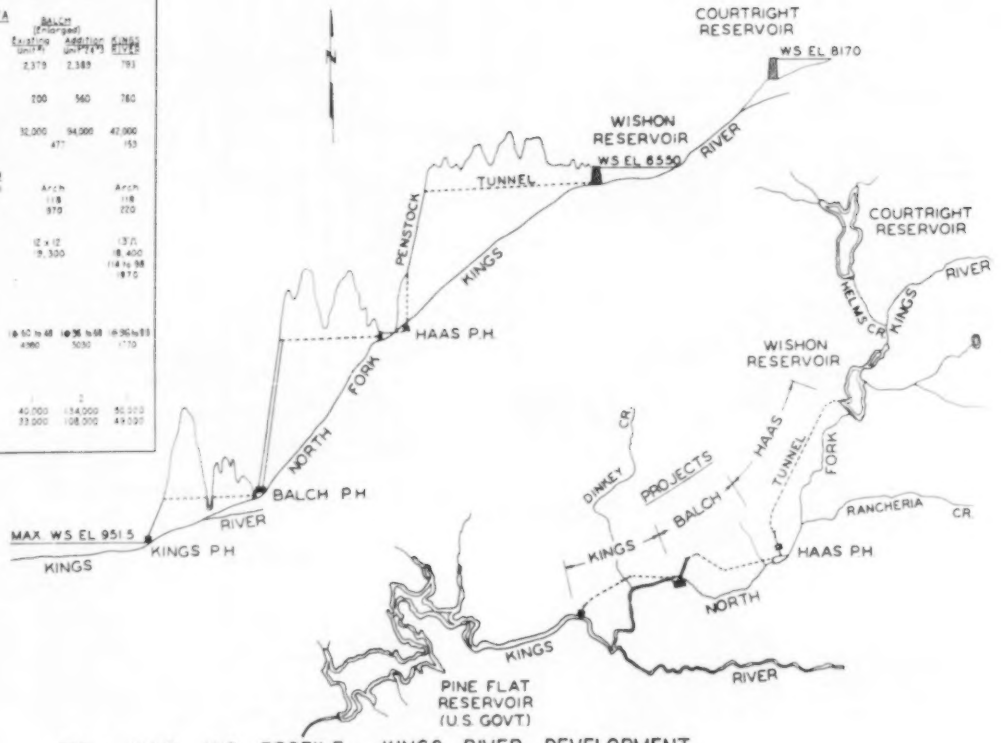


FIG. 1 MAP AND PROFILE - KINGS RIVER DEVELOPMENT

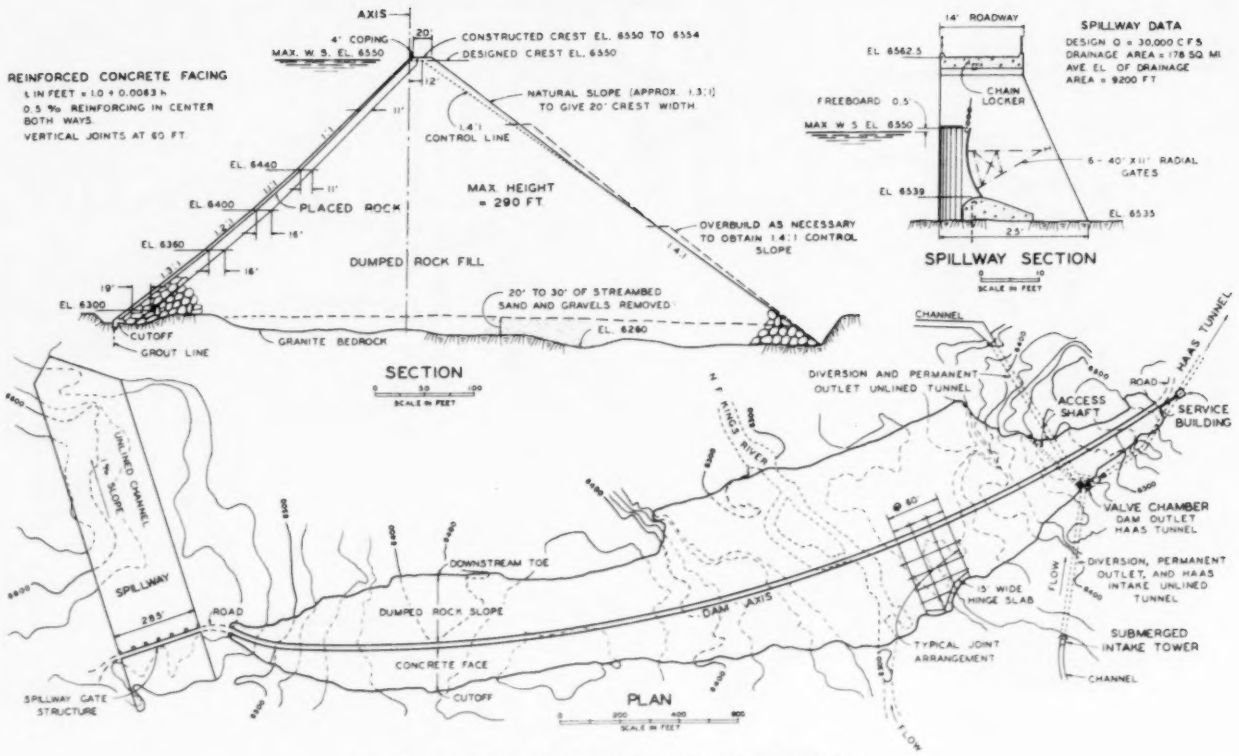


FIG. 2 WISHON DAM PLAN, SECTION AND SPILLWAY SECTION

construction and a review of other dams, resulted in a design and specification for Wishon that makes the concrete face rockfill competitive in cost with a thin earth core rockfill.

The main changes are: (a) steeper upstream face; (b) thinner placed rock; (c) easier placed rock specification; (d) the use of face lifts for construction access; and (e) elimination of keyways in the placed rock for the joint ribs. It is not the purpose of this paper to cover the dam design in detail, but it is important to note that these changes substantially lower the cost of the concrete face rockfill dam and are not considered to affect the high degree of safety of this type of dam.

The 290-foot high Wishon dam will have a crest length of 3350 feet and will contain 3.7 million cubic yards of quarried rock, faced with 60,000 cubic yards of reinforced concrete. It was intended to provide an ungated open crest spillway in the left abutment quarry for this high elevation unattended dam. The ungated crest spillway would have required a 5 foot higher dam than would a gated crest spillway. The cost of 5 feet of additional height for this long crested dam was $500,000 compared to $100,000 for the six 40 x 11-1/2 ft. radial gate spillway that was adopted. The height of the gates is small in order to use the wide quarry-spillway excavation and to have a minimum number of acre feet subject to gate control. The gates will be blocked open in the winter and an attendant will be necessary to top the reservoir off in the spring.

Diversion Tunnel

The size of the diversion tunnel, 15-ft. horseshoe, was to permit storage of 80,000 acre feet between the second and third construction seasons when there would be no spillway. Between the first and second construction seasons three floods of the 36 years of record would have raised the water level to the top of, and over the first year's rockfill. The rockfill would have been 90 ft. high with placed rock on the upstream face, but no concrete slab. Water would have percolated through the placed rock and about 1000 cfs would have overtopped the fill. Unfortunately, for rockfill dam experience, the 1956-57 runoff was less than an average water year.

Dam Outlet and Power Intake

Intake Tower. The intake tower, Fig. 3, is located at the upstream end of the diversion tunnel and serves as the permanent outlet for the dam and as the intake to Haas power tunnel. The 15-ft. square tower is capped by a 30-foot high square galvanized steel trash rack with 2 in. clear bar spacing. The gross trash rack area of 2000 sq. ft. gives an average velocity of about 0.4 fps for the 760 cfs power tunnel flow and 1.7 fps for the 3500 cfs outlet flow. The clear bar spacing of 2 in. is about the same as the minimum dimension of the nozzle opening. The 760 cfs will pass through 12 small nozzles having a do-nut shaped opening of 7.4 in. outside diameter and about 2-5/16 in. thick. The bar spacing should, perhaps, be less than the 2 in. but it has been found that pine needles can thoroughly plug trash racks with smaller spacing, and P. G. and E. has had no trouble with impulse wheel nozzles where clear rack bar spacing is about the dimension of the annular opening at the nozzle ring.

The top of the concrete tower is set at an elevation to give a pool of 1200 acre feet or less when the tunnel is unwatered for inspection or maintenance.

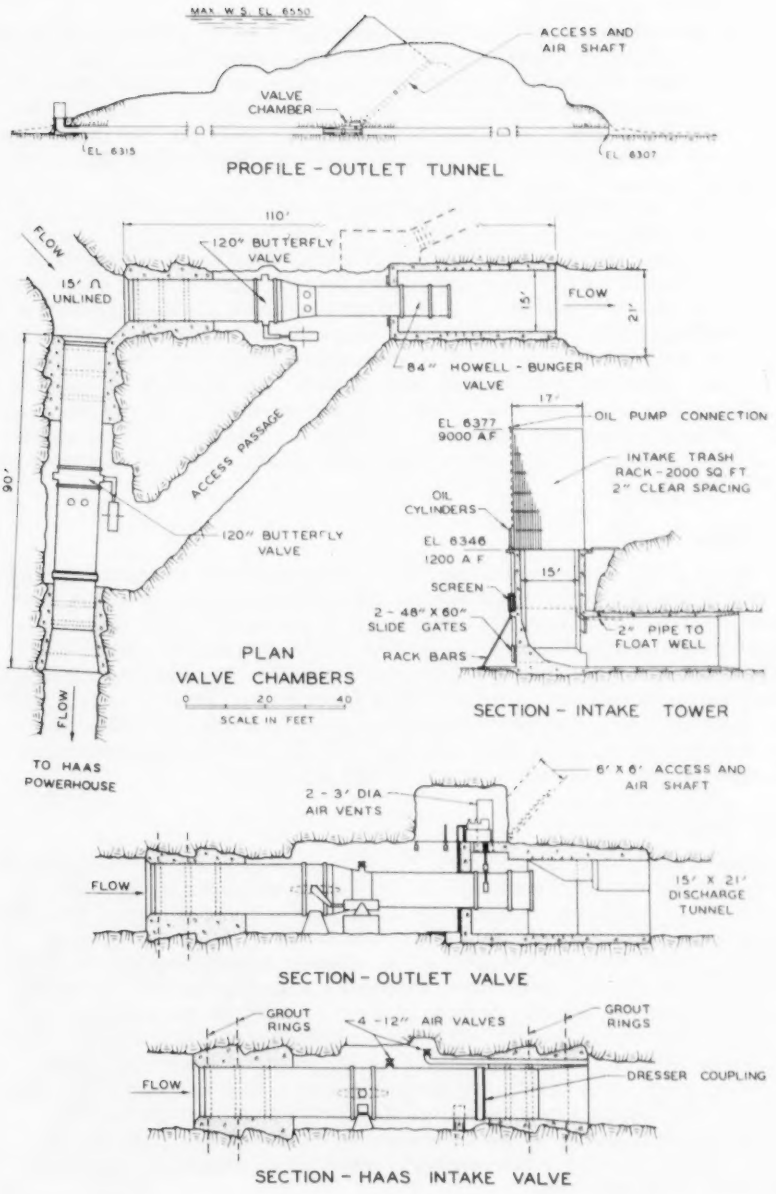


FIG. 3 WISHON DAM OUTLET WORKS HAAS INTAKE WORKS

Two 4' x 5' oil cylinder operated slide gates, normally closed, make it possible to draw water below the top of the concrete tower at 760 cfs. Closing these gates permits inspection of the unlined tunnel between the tower and the butterfly valves. The cylinders are located above the top of the concrete tower to permit their maintenance. The stroke of the operating cylinder is the gate height plus several inches and the cylinder is grouted in place with the gate properly seated and piston against end of cylinder to prevent jamming of the gate.

Dam Outlet Valve Chamber

A 3500 cfs capacity is required in the 1957-58 runoff period to permit storage of 80,000 acre feet in the partially completed dam that will have no spillway. The 80,000 A.F. capacity will store any high peak winter floods of record since they are of short duration for the high elevation drainage area. The snowmelt provides the sustained flow and on 3 of the 36 years of record, with outlet fully open, the 80,000 A.F. would have filled and about 1000 cfs gone through the rockfill. The 3500 cfs discharge will be more than necessary for future use and will permit flood regulation. Discharge in advance of spilling can appreciably reduce the peak flood flow and such has been done with the 5000 cfs outlet works at Salt Springs dam.

The use of an underground discharge chamber for a Howell-Bunger valve has been used by TVA at Watauga and Fontana. It happens that head and discharge requirements are the same at Wishon as at Fontana and the size of the Howell-Bunger valves is the same (84 in.). The Wishon discharge chamber is patterned after the Fontana installation. Fig. 3 shows the arrangement. A 6 x 6 ft. shaft inclined at 45° and 210 ft. long, serves as access and air vent. The 10 ft. diameter butterfly shut-off valve is oil cylinder operated. The trunnion mounted cylinder is preferred to a fixed cylinder with cross-head on the basis of being lower in cost, more easily set up and best from standpoint of simplicity for operation and maintenance. The piston rod is in the oil cylinder when the valve is in its normal position. Oil pressure is from a water motor oil pump (300 psi) or a gravity tank (100 psi) in the service house on the crest of the dam. The 100 psi pressure can be boosted to 300 psi by hand pump if necessary. It is considered desirable to be able to close both 120-inch butterfly valves from the service house in the event of trouble in the underground valve chamber. The Howell-Bunger valve is water motor operated. When the discharge valve is open, an ejector is operated to keep the valve chamber drained.

The unlined discharge tunnel is in excellent massive granite and is considered suitable for the turbulent flow as it leaves the discharge chamber. The original plan was for a 108 in. pipe to be on piers from the plug to an outdoor valve house. The diversion tunnel was already driven when the underground discharge scheme was adopted, and it was necessary to enlarge the 15 ft. tunnel section to ensure free flow in the tunnel. The underground discharge arrangement is lower in cost, eliminates maintenance and possible vibration of the high velocity discharge pipe and eliminates freezing problems.

Power Tunnel Intake

The intake tower and the unlined tunnel to the wye serve both the permanent dam outlet works and the power tunnel. The power tunnel intake,

Fig. 3, consists of a 10 ft. diameter butterfly valve in a dry chamber. Ten ft. diameter pipes are embedded in concrete plugs on each side of the valve. The valve is identical to the outlet shut-off valve. Both valves are designed to open or close under free discharge conditions. Access is from the outlet valve chamber. The butterfly valve is exceptionally reliable and the single valve controlling flow from the large storage reservoir is considered satisfactory. P. G. and E. has 95 butterfly valves in sizes ranging from 3 feet to 13 feet diameter and all give trouble-free service.

A dresser coupling is used between the plugs to eliminate longitudinal stresses that would be caused by temperature and Poisson's ratio. A draft tube taper is used on the discharge end to minimize head loss. Both the 120 in. butterfly valves can, in emergency, be closed from the service building on the crest of the dam. The air valves release air when the tunnel is being filled by the partially opened butterfly valve and admit air when the valve closes under flow conditions.

Reservoir Level Indicator

It is necessary to know the water level of Wishon Reservoir to 0.01 feet each day in order to allocate irrigation water on a basis of daily natural flow. The reservoir site is subject to severe freezing and heavy snow, and is inaccessible for about five months each winter. A 4 in. drill hole 270 ft. deep, from service house to tunnel, is used as a float well for a Stevens water surface detector. The bottom of the well is connected to a screened opening outside the outlet tower by a 2 in. galvanized pipe in the tunnel. In the valve chamber are blowdown valves to keep the 2 in. line clean. The detector is a 2-3/8 in. diameter x 14. in. float attached to a small gauge stainless steel wire which is controlled by a balanced lever arm mechanism. The wire is let in or out by a battery operated motor, actuated by mercoid switches on the lever arm. This mechanism, requiring neither a large float well nor a counterweight well, will make a low cost installation. The detector actuates both a recorder and a telemark radio which sends the water level to Haas Power House. It was planned to grout and redrill the 4 in. well to give a well that should not corrode, leak, or ravel; but perfect cores made it unnecessary to grout.

Power Tunnel

Summary. The power tunnel is a 13 ft. unlined horseshoe section, Fig. 4, 6-1/4 miles long. The rails and ties are removed, but no mucking of invert is required. There is one adit which has a 7 ft. square sliding door in the plug for future access. Upstream from the surge chamber is a large storage sand trap and downstream, a steel liner to the portal, Fig. 4. Where support is required, a concrete horseshoe or circle will be used. Soft seams that would wash and later create rock falls, are mined 6 inches deep and plugged by gunite.

Alignment and exploration. Because of the high head, 240 to 330 ft., resulting from the storage reservoir and long length of tunnel, core holes were drilled at points that would control alignment. At four creeks holes were drilled from surface elevation of 6550, maximum Wishon reservoir water surface, to tunnel grade. It was considered that if these holes showed good rock the tunnel alignment would be such that a minimum depth of cover of

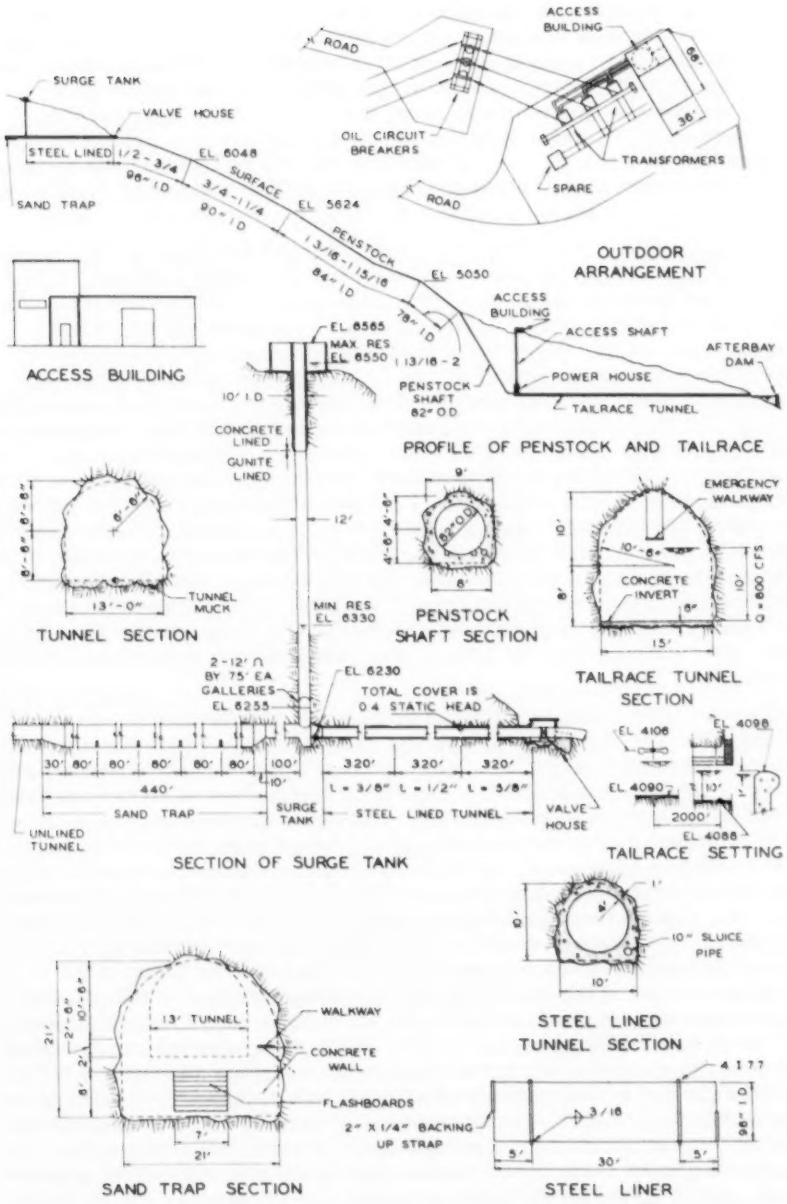


FIG. 4 HAAS PROJECT - PROFILE AND SECTIONS - FROM SAND TRAP TO AFTERBAY

static head in feet would be used. To move the tunnel further into the mountain would increase its length. The upper portion of the holes showed hard granite with seams and some extensive zones of soft grained granite. At tunnel grade the rock was sound and the alignment of total cover equal static head was accepted. Lateral cover at these creeks is three times static head. An exception to the criteria was made at a deep draw near Wishon dam where the surface granite was so sound that no drill holes were put down and cover of 180 ft. where head was 280 ft. was used to save tunnel length. It is not intended to concrete line this zone, but if it is decided to be necessary, after tunnel is driven and inspected, the saving in length of tunnel is so great that it would pay for lining.

Unlined section. It has been P. G. and E.'s practice to use unlined tunnels where rock conditions permit because of the major saving involved. Tunnel and turbine operating experience of 800 mile-years of unlined tunnel service has been very satisfactory. Rock falls in early tunnels were due to not plugging soft seams. Water would wash out the seams and cause large blocks of rock to fall in the tunnel. No rock falls have occurred in tunnels constructed since 1947 where proper dental work has been done during construction. An unsupported tunnel lined with concrete costs approximately 50% more than an unlined tunnel of equal head loss. This conclusion is based on "n" = 0.015 for concrete and "n" = 0.030 on design area for the unlined section; unlined sizes from 11 to 26 ft. horseshoe; and on bid prices. The value of "n" for new concrete may be 0.012, but if the water acts on the mortar it may reach 0.015 with time. The additional excavation for the larger unlined tunnel is at a low incremental cost compared to the cost per cubic yard for the smaller lined tunnel. Since the unlined tunnel costs less for a section of equal head loss the economic size is larger and of less head loss. More effective head is economically developed by using an unlined tunnel.

Unlined invert. None of P. G. and E.'s unlined tunnels have a concrete lined invert since the cost cannot be justified. The lined invert would save the cost of a large storage or sluiced sand trap and would earn power benefits due to improved hydraulics, but not enough to justify its cost. The intangible benefit of the improved roadway in the event of maintenance is given no dollar value since an unlined tunnel with proper dental work on seams should require no maintenance. Also, the "unmucked" unlined invert of a pressure tunnel provides a waterbound macadam surface that can be traveled if necessary. The important requirement for maintenance, should it ever be necessary, is a large access door in the adits. The improved hydraulics of the lined invert section is not very great since the muck on the invert is the smoothest portion of the unlined tunnel. The weighted effect of the relative roughnesses is considered to be proportional to "n" squared, using Mannings "n." P. G. and E's design value of "n" = .030 on design area is based on head loss measurements in a number of tunnels and is perhaps a composite "n" of unlined walls and waterbound macadam invert with possibly "n" = 0.032 on walls and 0.024 on invert. For concrete lined invert "n" may be assumed to be 0.015. The 0.015 value for screeded invert concrete must include increased roughness due to erosion and to rock spalls that will collect on the invert. Accepting the above values, a comparative composite value of "n" for a lined invert is computed to be "n" = 0.028. Using this "n," and allowing for reduced area due to a 6 in. invert, the reduction in head loss for the lined

invert is 10%.

An estimate of the net saving in not lining the invert of Haas tunnel is as follows:

Invert lining (31,270 ft. @ $18)	$563,000
Sand trap (as bid, less cost of equivalent length of tunnel)	66,000
Direct Cost Capital Saving	$497,000
Appropriation saving (1.15 direct)	574,000
Penalty for head loss (.10 x 60' x $14,000. $14,000 is the capital value of 1 ft. of head after the increased cost of the power plant has been paid for.)	84,000
Net Capital Saving	$490,000

Hydraulics. Before driving the tunnel the hydraulics must be based on a design area since the percent overbreak is not known. The contractor drills several inches outside this line to avoid the cost of removal of rock points at the lesser cost of slightly increasing mucking. This results in average overbreak of about 6 to 9 inches, or overbreak area of some 10 to 25 percent. P. G. and E.'s tests on unlined tunnels give values of "n" on measured areas of 0.029 to 0.035 and on design areas of 0.024 to 0.029. The smaller values on the design area are due to the overbreak area. A value of 0.030 on design area was used for economic studies and for determining the plant capacity for Haas. For setting tunnel grade and for computing surge chamber load-on, a very conservative value of "n" = 0.035 is used so that design flow or greater can be carried without any question.

Storage sand trap. A large storage sand trap, without sluice, is used to collect the sand and gravel that will travel down the tunnel from the unmucked and unlined invert. The trap, Fig. 4, will store 2600 c.y. of material which is equivalent to a 2.2 in. depth of muck on a 12 ft. width of invert 31,270 ft. long. The trap is divided into 5 sections to insure that it will all be effective in storage. The 6 fps tunnel velocity is reduced to 4 fps over the dividing walls. The trap may not fill. If it does, a 12 in. sluice from the downstream section will indicate the presence of sand or gravel in the downstream section. If cleaning is required, the fine material from the downstream sections will be pumped out through this 12 in. line on a scheduled shutdown.

The large storage trap was adopted in preference to a small trap with a sluice on the basis of lowest capital and operating cost and better performance. Much of the drainage channel from the tunnel portal is in erodable decomposed granite and regular sluicing would cause severe erosion or require about one and one-half miles of pipe. Also, sluicing from a pressure conduit does not move much material each time, is not always satisfactory and requires operation and maintenance. It has been found that the surface muck, to about 2 in. dimension, moves down the tunnel invert during the first year of operation. The larger sized muck then retains the remaining small material and there is little sand or gravel that moves after the first year.

Steel liner. A 96 in. diameter by 960 feet long steel liner was used from surge chamber, where cover equals static head of 330 ft., to the portal. The

economic penstock diameter at the portal is 96 in. Actually, the theoretical economic diameter of a steel lined tunnel is less than 96 in., but it is not considered reasonable or practical to make it smaller. The tunnel is driven to a 10 ft. section rather than the 13 foot horseshoe unlined section. The liner is designed for full head to a point where cover is 0.4 static head with stress of 12,500 psi, 1/4 ultimate for A285B plate. The 5/8" plate thickness is decreased to 1/2 in. and 3/8 in., Fig. 4, which results in free pipe stresses of 15,500 psi and 20,800 psi for increased cover. If supports had been required the 3/8 or 1/2 in. plate pipe would have been backed up by reinforcing steel hoops since the plate was on order before the tunnel was driven. Only 160 ft. from the portal required timber support and no reinforcing hoops are being used. The 5/8" plate pipe sections are radiographed and stress relieved; the 1/2" and 3/8" thick pipe sections are not. Four inch I stiffeners are used at 20 ft. spacing to stiffen the pipe during shipping and erection, and to localize buckling that may occur due to outside water pressure. The crown concrete to rock contact and any hollows between liner and concrete will be grouted to minimize the danger from external water pressure. Field joints will be single butt welds from the inside with backing up strips, Fig. 4. Sections are limited to 30 ft. length because of the mountain road to the portal. The interior will receive a shop spun coating of hot coal tar enamel and an exterior spray coat of white wash to protect the shop enamel from excessive heat of the sun during shipping.

Surge Chamber

The Haas plant is a system governing plant and designed for load-on or off in one minute. This time would not be rapid enough for governing local load or a small system, but for P. G. and E.'s large integrated system, a 1-minute plant is suitable and does not require a premium to be paid in penstock water-hammer. A 10% value is used as a minimum. In dropping load at a rate faster than full load in 1 minute, the jet deflector will operate in 2 seconds and a 1-minute needle closure keeps pressure rise below 10% and minimizes loss of water.

Two 300 ft. deep diamond drill holes indicated that the formation would be fractured and poor for the upper half of the surge tank. It is planned to concrete line, and reinforce for full head, the upper 50 ft. of the surge shaft. Concrete with nominal reinforcing or gunite lining, as determined by rock conditions, will be used for the remaining 250 ft. A drill hole in a creek 2400 ft. upstream indicated ideal massive granite and an unlined surge chamber at that location was studied. The cost for this upstream location due to increased waterhammer and due to the lower tunnel portal, exceeded the cost of full concrete lining for a surge chamber near the portal.

The surge chamber type might be described as a gallery-simple-differential design, Fig. 4. The unusual design is occasioned by the 220 ft. operating range of the storage and diversion reservoir, by the very high head plant (2444 ft.) and by the cost of a foot of head on the long (4600 ft.) penstock. The galleries provide minimum down surge for load-on from minimum reservoir level. The high head on the unlined tunnel is undesirable and both the use of galleries, and taking one-minute load-on from reservoir level 6366 rather than from minimum storage level of 6330, give minimum pressure at the downstream end of the tunnel. For rejection from maximum reservoir level the tank will act as a differential tank with most of the water spilling

over the riser. The diameter of the rejection tank is such that the sum of tank cost plus incremental liner and penstock cost is a minimum. The surge above static is only 15 ft. for the 6-1/4 mile tunnel and 1 minute rejection. The ports are small since they aren't necessary for load-on and this gives an efficient differential design for rejection. The surge chamber is located so that the large rejection tank is in open cut to avoid the higher cost of underground excavation. The steep sidehill slope did not make possible a still lower cost open cut basin. For intermediate reservoir levels the galleries or tank may not come into action and surges in the simple shaft may be as great as 110 ft. for a 1 minute load change. The economy in the small diameter shaft is possible because of the very high head plant and such large surges, being a small percentage of the effective head, should not interfere with the governing.

Penstock Valve

It was decided to use a valve, 96 in., at the top of the penstock since there is only one valve at the tunnel intake and it is inaccessible for some five months of the year. The penstock valve will be operated by an oil pressure trunnion mounted cylinder and will close automatically on 10% excess flow. Four 12 in. diameter air valves are used just downstream from the valve. The air valves are cast steel with bronze seats and stainless steel stem.(10) Oil will be provided from a 300 psi accumulator set. The valve house will be concrete block walls with flat concrete roof supported on steel beams.

Surface Penstock

P. G. and E.'s penstock practice(11) has been used in the design of Haas penstock. Three changes from earlier practice, made at Haas and also at Balch, were: the increase to 17,500 psi working stress, the use of 20% minimum efficiency riveted joints, and the painting of exterior aluminum coating in the shop. A thorough study of stresses, occasioned by the large tonnage (7000 tons) of penstock to be required at Haas and Balch, resulted in increasing the working stress above that used on previous jobs. The adoption of 20% efficiency joints permitted the use of two rows or rivets, rather than three or four, in connecting the heavy walled pipe. With the use of an expansion joint between anchors the longitudinal stress is limited to the friction on piers, deadend pressure on expansion joint, and bending moment. Though the joints are located at points of zero moment the joint is designed for the moment over the support to give a measure of resistance to moment in addition to the other forces. The joint is designed for the above loads or 20% minimum efficiency, and the 20% governs for the heavier walled penstock. Changes in diameter are made in bends to eliminate longitudinal stresses, and minimize head loss and cost.

The interior shop spun coal tar coating is very important from the standpoint of hydraulic economy since the head loss is 40% less than for a hand daubed coal tar ("n" = 0.010 as compared to "n" = 0.013). At Haas the use of spun coal tar provides $400,000 more net capitalized power value (28 ft. at $14,000) than a hand daubed coating and costs less. Net value refers to the value after the units and powerhouse have been enlarged to produce the power. The shop spun coating, including sand blasting and priming, is about 30 cents per sq. ft. as compared to about 60 cents per sq. ft. for a field hand daubed

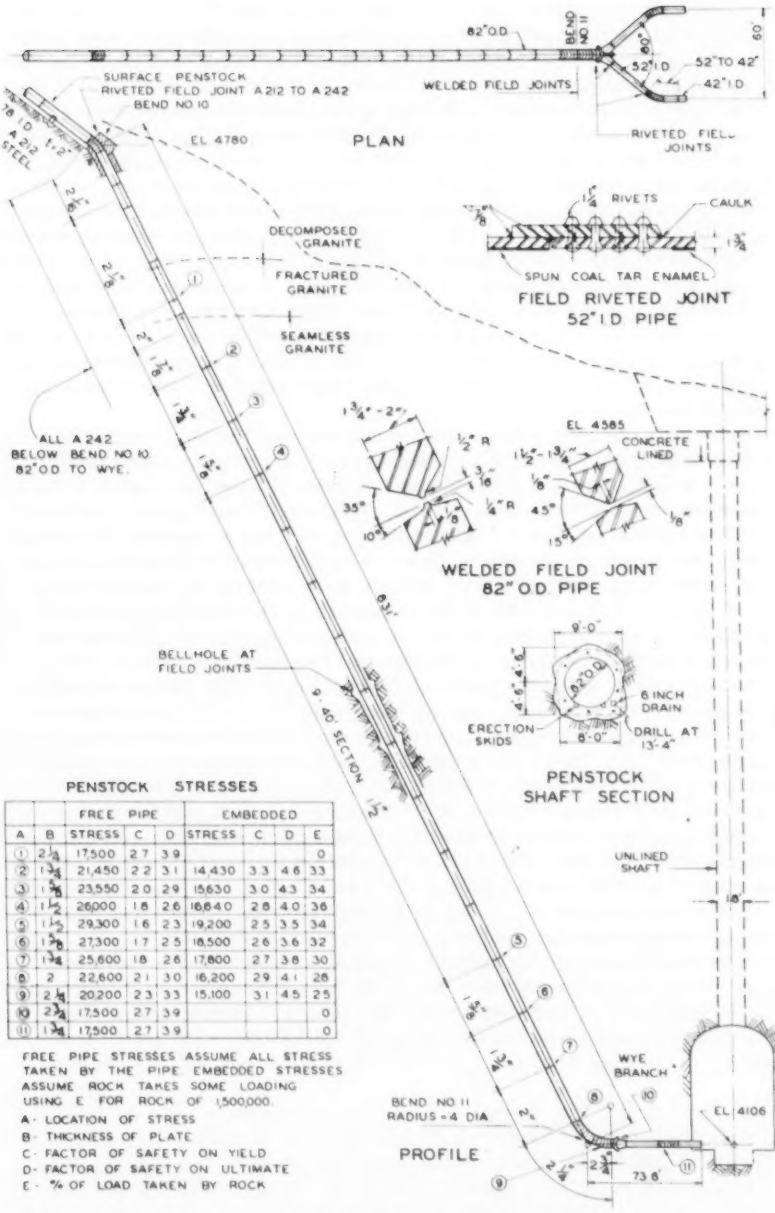
job. For Haas the cost difference is about $30,000 which is important but not as great as the hydraulic saving. The pipe in shipping will be subject to very warm weather in the San Joaquin Valley and to minimize the possibility of heat damage to the enamel the penstock is given a shop coat of aluminum over the shop sand blasted and red lead primed pipe. Again the shop exterior painting is much lower in cost than field painting, and damage in shipping is nominal.

Underground Penstock

The underground penstock, Fig. 5, consists of 760 ft. of 82-inch OD pipe embedded in concrete in a shaft 65° to the horizontal, a 76.5 x 52 x 52-inch symmetrical wye at the base of the shaft and two 52-inch, 60 ft. long branch lines to the units. The head at the top of the shaft and at the bottom are 1500 and 2344 feet respectively. This high head pipe in rock cover of nominal depth but excellent quality is not a normal problem of concrete backfilled steel liner or penstock design. It was decided not to share load with the excellent granite but to use somewhat higher free pipe working stresses. The embedded pipe is not subject to being hit by rocks, to high temperature changes and to such high secondary stresses as for a surface line, and the granite will certainly take some stress.

The criteria used for determining design stresses in the embedded penstock are rather arbitrary. The table of Fig. 5 shows the stresses for free pipe and also for embedded pipe. The embedded stresses are based on equal radial movement of the shell and the rock. The properties of the A242 steel for this penstock are 47,000 psi yield and 67,000 psi ultimate. Conventional surface penstock stresses were used through the decomposed and weathered surface granite and at the lower end from the wye to the units, including the wye. After entering sound seamless granite, thicknesses were decreased 1/8 inch per 40-foot section until the safety factor of free pipe was 2.6 on ultimate which for A 242 steel is 1.8 on yield. By keeping a uniform thickness of 1-1/2 inches to point 5 of Fig. 5, the stresses were increased, as rock cover increased, to 2.3 on ultimate which is 1.6 on yield for a free pipe. Between point 5 and the wye, the stresses are decreased because of proximity to the chamber and as a transition to normal stresses at the wye. With 25 to 35% of the load taken by rock, Fig. 5, the minimum safety factor is computed to be 3.5 on ultimate and 2.5 on yield. The design is perhaps conservative, but it is observed that this section of penstock is subject to very high head and not much rock cover. The saving in plate for the adopted design over a free pipe line with surface penstock design stress of one-fourth the ultimate is 210 tons or about $150,000.

Welded field joints were adopted for the embedded 82-inch OD line in the shaft. The riveted field joints of 20% efficiency as used on the surface line would be weak links due to the stresses of Poisson's ratio, temperature contraction and concrete setting. A 50% or greater efficiency riveted joint would take 4 or 5 rows of 1-1/4 inch rivets for the thick 1-1/2 to 2-1/4 inch pipe. The riveted joint in the steep shaft would be difficult and would be expensive both in the joint cost and in the concrete backfill for a larger bell hole. The welded joint was selected on the basis of being most suitable from the standpoint of its giving a high roundabout joint efficiency and lowest cost. A 285 and A 212 steels were eliminated due to the excessive thickness required of A 285 and the undesirability of the non-stress relieved field weld on



PENSTOCK STRESSES

		FREE PIPE			EMBEDDED			
A	B	STRESS	C	D	STRESS	C	D	E
①	2 3/4	17,500	2.7	3.9				0
②	1 3/4	21,450	2.2	3.1	14,430	3.3	4.6	33
③	1 5/8	23,550	2.0	2.9	15,630	3.0	4.3	34
④	1 1/2	26,000	1.8	2.6	16,640	2.8	4.0	36
⑤	1 1/2	29,300	1.6	2.3	19,200	2.5	3.5	34
⑥	1 3/8	27,300	1.7	2.5	18,500	2.6	3.6	32
⑦	1 3/4	25,600	1.8	2.6	17,800	2.7	3.8	30
⑧	2	22,600	2.1	3.0	16,200	2.9	4.1	28
⑨	2 1/4	20,200	2.3	3.3	15,100	3.1	4.5	25
⑩	2 3/4	17,500	2.7	3.9				0
⑪	1 3/4	17,500	2.7	3.9				0

FREE PIPE STRESSES ASSUME ALL STRESS TAKEN BY THE PIPE. EMBEDDED STRESSES ASSUME ROCK TAKES SOME LOADING USING E FOR ROCK OF 1,500,000.

A - LOCATION OF STRESS
B - THICKNESS OF PLATE
C - FACTOR OF SAFETY ON YIELD
D - FACTOR OF SAFETY ON ULTIMATE
E - % OF LOAD TAKEN BY ROCK

FIG. 5 HAAS PENSTOCK – UNDERGROUND PORTION

thick A 212 plate. A review of possible steels resulted in selection of Type A 242 of Mayori R, Kaisalloy or Tri-Ten quality with chemical composition directed toward the properties of ductility and weldability.

Double butt field welds, Fig. 5, were chosen in preference to single butt welds. A single butt weld from the inside would be more easily accomplished and eliminate the bell hole cost. The double butt weld will cost more but provide a weld more suitable to radiographing and more acceptable as a non-stress relieved weld. Preheat of 250-300° F by electric pads is specified and may have some stress relieving effect. The shop spun coal tar is stopped 2 feet back from the joint to permit preheat and welding.

The wye field joints and the joints in the 52-inch branch pipes are field riveted, which may seem inconsistent, but has been done for several reasons. The wye and branches are adjacent to the powerhouse chamber and all factors in design and installation should be conservative. It is considered undesirable to field weld to the very rigid and heavy wye. The pipe is horizontal, excavation gives several feet of working room for other reasons and the butt strap fitting up and riveting may be done conveniently and at reasonable cost. The 50% efficiency riveted penstock joints between the wye and turbine are preferred and are convenient.

The 82-inch OD pipe is to an outside diameter so that erection skids, drainline and all installation clearance can work to the line of the outside of the pipe. Shop fabrication is in 40-foot sections, the heaviest being 32 tons. All welds are double butt welds and radiographed. The completed sections are stress relieved and then coated with spun coal tar enamel on the inside. The exterior will be embedded in concrete and is coated with white wash to minimize the danger of the heat of the sun to the coal tar during shipping. Much of the equally heavy walled Balch penstock has been fabricated, and some of Haas, and a maximum difference between maximum and minimum diameter of 1/16 inch is obtained for the end of a section.

The wye is of A 242 plate with shell thickness of 2-3/4 inches and ring and clamp plate thickness of 4 inches, maximum for A 242 plate. The cylindrical and cone butt welds are radiographed and fillet welds magnafluxed. Pressure test is to 1530 psi, one and one-half times design stress. Guide plates are used in the crotch of the wye to make the area of each branch change uniformly and minimize wye head loss. In making the Balch wye one crack developed in the middle of the 5-inch plate ring clamp as the last cone section was welded. Procedure was to weld the large diameter section to the ring clamp then one cone leg to ring and C-clamp and then the last cone leg. Carbon arc gouging of the crack, stress relieving, re-welding, stress relieving and pressure testing completed the job. At Haas the schedule was very tight and all pieces were welded at the same time. Several cycles of gouging and stress relieving were required. The cracks, several feet long, occurred in the 4-inch plate adjacent to the weld and several inches deep. Both a spherical and a pantleg wye were bid. All bidders submitted lower prices for the pantleg wye, prices 60 to 70% of the lower weight spherical wye.

The 6-inch drain, Fig. 5, may not actually be required. The thick walled 82-inch OD pipe is in no danger of collapsing. The thought in having the drain, which discharges visibly in the power house, is to warn of any penstock leak which might be a hazard to the power house chamber. The six-inch size is large to minimize the possibility of plugging and is located in a part of the excavation where there is room for it. The top is accessible so

that the drain may be cleaned and may be tested.

The penstock backfill concrete will be 2000 psi concrete with low heat cement. Mix design will be to minimize shrinkage. Two tapped and threaded 1-1/2 inch holes, opposite each other at 10-foot spacing, are provided for grouting between steel and concrete and between concrete and rock. The tapped 1-1/2 inch holes will be plugged and seal welded after grouting. The drain holes will then be diamond drilled through 1-1/2-inch tapped holes in the shell and into the 6-inch drain and on until one foot is into rock.

The design size of the shaft, Fig. 5, gives 13 in. clear distance from pipe to rock. Bell holes at field joints are required for welding and radiographing. A horseshoe shape provides room for the rail skids, the drain line and a manway along pipe for access to joints and for a man to vibrate concrete. Minimum thickness of concrete is desirable both from the standpoint of cost and shrinkage.

Underground Plant Design

General

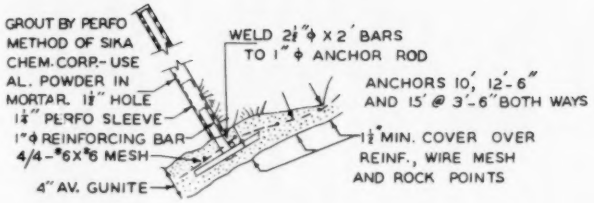
The underground power house will be approximately 500 feet vertically below the surface and 2000 feet from the river. The general arrangement and details are shown on Figs. 4, 6 and 7. The penstock will be in a 760-foot shaft leading to the power house chamber. The chamber, which houses valves, units, auxiliaries and control room, is 173 by 56 feet in plan and 100 feet high. An 18-foot diameter access shaft will lead to the surface and contain generator leads, elevator, stairway, piping and control leads and serve as the exhaust ventilating duct. The 2000-foot long, 17.5 x 15-foot wide unlined tailrace tunnel will serve as construction and future access for heavy loads, as well as an emergency personnel access and ventilating duct.

Before discussing details of the underground plant, some of the studies and considerations leading to the adopted design will be reviewed.

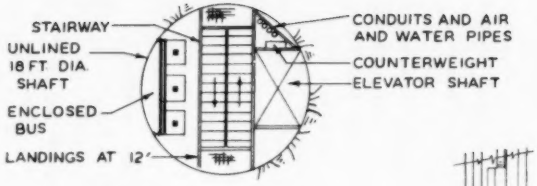
The Haas plant was originally laid out as a surface scheme and on essentially the same alignment as the adopted underground scheme. The surface layout was the most economic of five alternate schemes, but the high cost and head loss of the heavy walled penstock (2 to 3 in. thickness) on the flat profile at the lower end was discouraging. A study of an underground layout was made and indicated that the underground layout was substantially lower in cost and developed 30 feet more effective head. Even if the chamber excavation were to cost several times the estimate, it was still economic to go underground. With such a margin for contingencies, and with nominal exploration, it was decided to go underground for the purpose of effecting capital savings and increasing power values. Additional inherent benefits are: development of maximum effective head, conservation of 2500 tons of steel, a safer and more permanent installation, and minimum maintenance.

Exploration

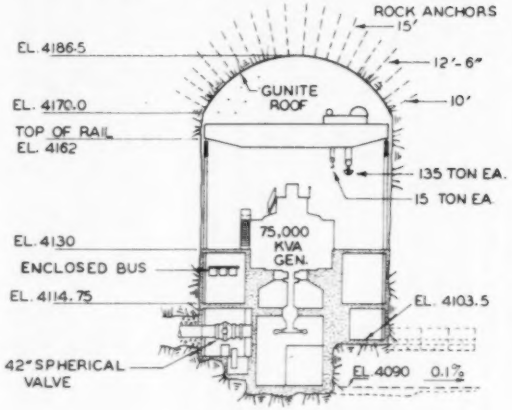
It was decided to put down an NX core hole, 500 feet deep, at each end of the proposed chamber. The first hole, DH #1, provided essentially all 10 foot cores of ideal granite. The second, DH #2, 200 ft. away, was a perfect hole until the elevation of the chamber was reached, where some 50 ft. of close cleavage planes and soft seams, in a plane 20° to the horizontal, were


GROUT BY PERFO METHOD OF SIKA CHEM. CORP. - USE AL. POWDER IN MORTAR. 1¼" HOLE
1¼" PERFO SLEEVE
1" ϕ REINFORCING BAR
4/4-#6X#6 MESH
4" AV. GUNITE
WELD 2½" ϕ X 2' BARS TO 1" ϕ ANCHOR ROD
ANCHORS 10', 12'-6" AND 15' @ 3'-6" BOTH WAYS
1½" MIN. COVER OVER REINF., WIRE MESH AND ROCK POINTS

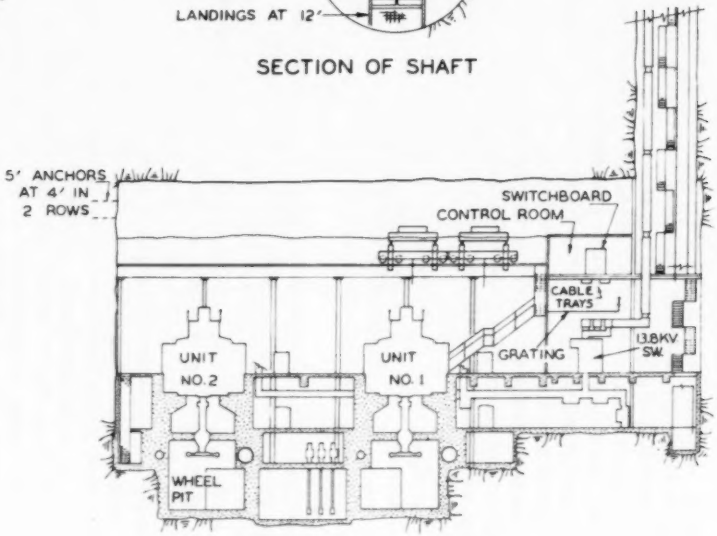
ANCHOR AND GUNITE DETAILS



SECTION OF SHAFT

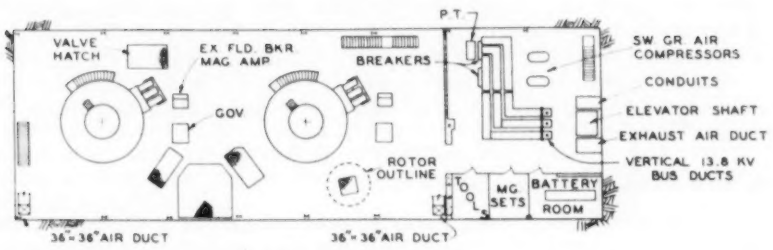


TRANSVERSE SECTION

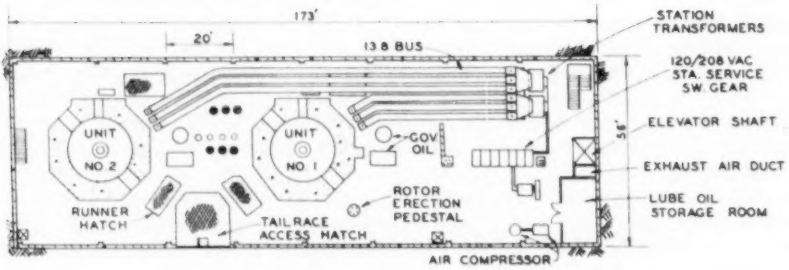


LONGITUDINAL SECTION

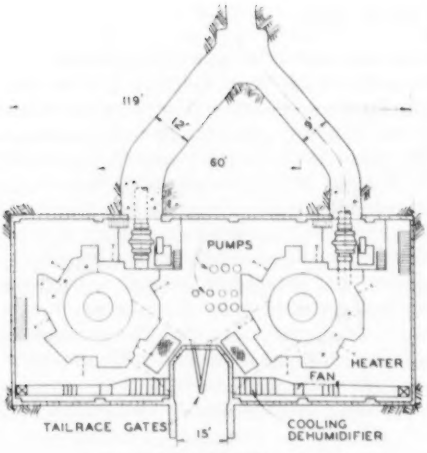
FIG. 6 SECTIONS AND ROOF DETAILS — HAAS POWERHOUSE



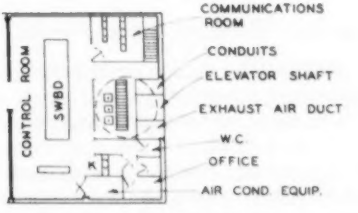
MAIN FLOOR PLAN



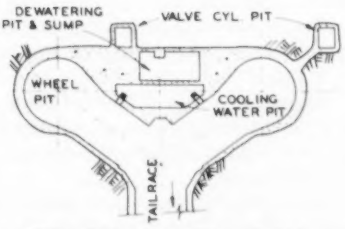
BASEMENT PLAN



SUB-BASEMENT PLAN



SECOND FLOOR PLAN



TAILRACE AND PIT PLAN

FIG. 7 HAAS POWER HOUSE - PLANS

encountered. DH #2 was getting away from the ridge and toward a nearby creek which may have been in a faulted zone. DH #3 was started a distance equal to chamber length, 170 feet, from DH #1 and was essentially a perfect hole and of the same grain structure granite as DH #1. With an excellent core at each end of the then relocated chamber, it was decided to go underground and do no more drilling.

Economics of Going Underground

In deciding to go underground, a capitalized saving of about $1,600,000 was estimated. All contracts have been let and the excavations, which contain the major contingency, have been completed. Though no troubles have been encountered in the excavations, the excavation costs have been higher than estimated and the actual saving over a surface scheme appears to be on the order of $850,000.

The capital and power value benefits in going underground are in the conduit from the point where it enters the penstock shaft to the afterbay. The 3100 tons of penstock required for this portion of the surface penstock, at $900 per ton installed, would have cost about $2,800,000. The installed cost of the underground penstock ($1,050,000) plus the tailrace tunnel ($600,000) will be about $1,650,000, which gives a conduit saving of about $1,150,000. The underground power plant, consisting of power house, access shaft, service building, and switchyard, will cost about $750,000 more than the comparable installation for the surface scheme. The net direct capital saving is about $400,000. In addition to the capital saving the capitalized value of the increased power output due to 30 feet more of head is $450,000. The net saving in being underground is on the order of $850,000.

Optimum Location and Alignment of Power Plant

It has been demonstrated that the higher cost of the underground power house and access shaft was more than paid for by the savings in the flat section of penstock of 2 to 3-inch shell thickness. The top of the penstock shaft is at the bottom of a steep surface penstock slope. Any further shortening of the surface penstock, by moving the power house further into the mountain, would not decrease penstock length very much and the penstock saved would be of thinner plate. The cost of the increased length of penstock in shaft, and of the longer access shaft, would have exceeded the saving in surface penstock. The optimum location was rather closely defined by the topography.

The surface penstock, as is customary, was located on a ridge to keep the tonnage at a minimum. For the initial underground layout the penstock dropped off the ridge steeply to the top of the penstock shaft. The relocation of the power house directly under the basic ridge as dictated by the diamond drill holes, required a somewhat longer penstock shaft, access shaft, and tailrace tunnel. The optimum paper location was thus changed slightly by the exploration.

Having the power house and tailrace tunnel located, the penstock shaft alignment was then firmed up. The PI of the wye and horizontal penstock adjacent to the power house chamber was arbitrarily located 70 feet upstream from the power house wall in order that the wye and branch excavations would not undermine the high chamber wall. This leaves a substantial block of rock, Fig. 7, to support the wall during construction. Later the penstock will be concrete backfilled to the wall of the power house chamber. The

economic slope of the penstock shaft was determined to be 65° to the horizontal. Even using higher stresses for the penstock in the shaft the cost of shaft and concrete backfill made it apparent that a minimum amount of penstock should be underground. Alternate studies showed that between 90° and 65° to the horizontal the annual costs were about the same within the accuracy of the estimates. The 65° slope was selected in preference to 90° since at the same annual cost it gave slightly more power. A 50° slope, which might be very appropriate if the penstock were already underground, was not economic here.

Some Basic Decisions in Underground Layout

It was recognized that the cost and possible contingency in the excavations was a major factor of uncertainty in an underground power plant. The main criterion in selecting a layout was to have minimum excavation.

The most significant decision in basic layout was to use the tailrace tunnel for construction and permanent access with a minimum size vertical shaft. The use of the tailrace tunnel for permanent heavy maintenance access makes it possible to accommodate all other access requirements in a small diameter vertical shaft. The cost of a high capacity crane at the top of a large diameter shaft or a separate access tunnel is avoided. This multiple use of the tailrace tunnel requires that the plant be shut down when loads greater than 2-1/2 tons (shaft elevator capacity) must be handled, and that flood flow be such that it is possible to enter the tunnel. Access for heavy loads is seldom required and two large upstream storage reservoirs provide substantial control of the river. The capital cost saving in using the tailrace tunnel as the access tunnel, is considered to greatly outweigh the possible but improbable impairment of operation. The large size and growth of the Pacific Gas and Electric system is such that reasonable operating risks are increasingly more acceptable than the expenditure of money.

A brief study showed conclusively that the transformers should be above ground. The 500 feet of isolated bus in the shaft is expensive but appreciably lower in cost than the underground alternative which requires underground excavation and high voltage cable.

Two independent means of personnel access were considered necessary. The access shaft with elevator and stairs is the primary access. A small 45° stairway shaft at opposite end of the chamber was considered but abandoned in favor of a walkway in a Gothic arch tailrace tunnel. The 2000-foot hanging walkway, Fig. 4, was estimated to cost less including the incremental cost of the larger tailrace tunnel. The additional space is useful in plant ventilation and insures free flow conditions. The emergency walkway may be awash but usable during flood conditions.

The underground estimate, used in comparison with the surface layout, included a concrete arch ceiling in the power house, a concrete block curtain wall in event of dampness and other items that were not intended to be used unless essential, but were included to allow for contingency. Fortunately, the very favorable dry and safe rock conditions have permitted economies over the estimate which have helped to offset the higher than estimated excavation costs.

Power Plant Equipment

Turbines. The selection of two 92,000 hp vertical-shaft multinozzle

impulse turbines was straightforward. The criteria for number of units for the large integrated P. G. and E. system is the largest economic and practicable size. 400 rpm, six nozzle machines with integral cast runners are being furnished by Pelton Waterwheel Company. The turbines will operate with a static head range of 2444 to 2224 feet. The design 92,000 hp is for Q = 792 cfs and static head 2444 (effective 2324). The dependable capacity is for minimum level and same nozzle stroke which delivers 760 cfs. The vertical shaft multinozzle turbine was selected over a horizontal shaft unit for a number of reasons: higher efficiency, fewer units, nozzles may be set closer to tailwater, higher speed and therefore smaller turbine and generator, effective energy dissipation into wheel pit walls, less hazard due to wheel or bucket failure, and less water hammer hazard to the penstock from a single faulty nozzle operation because each nozzle controls a small part of the flow. The turbine distributor casing will be tested to 1.50 times working pressure both in the shop and in place. Concrete will be poured with no pressure in the casing.

Turbine valve. The turbine valve will be of the straight flow spherical type. The economic diameters of the penstock leading to valve, the valve, and the distributor casing entrance are 52, 42 and 46-1/2 inches respectively. Bid prices on valves of 42 and 46-1/2 in. size indicated that the 42-inch valve was economically appropriate. The valve is double seated with normal closure of the downstream seats and upstream closure to permit maintenance and adjustment of the downstream seat. A Dresser Coupling is located between the valve and distributor casing to control longitudinal pipe stresses and permit disassembly.

Generator. Two Westinghouse 75,000 kva, 0.9 pf, 440 rpm, 60 cycle, 3 phase generators are being furnished. An overhead thrust bearing will support the rotating parts and the generator and turbine guide bearings will be capable of carrying the maximum side thrust caused by the hydraulic unbalance of any combination of nozzles. Generator cooling water will be pumped from the tailrace.

Governor. The Pelton Waterwheel Company's governors will be an actuator type and will control the needles and jet deflectors. The needles will be operated by individual servomotors and the rate of closing and opening will be set for one minute. The needle servomotors will be provided with external springs to automatically close the needle nozzles in the event of the loss of governor oil pressure. In the event of sudden load rejection, deflectors will deflect the jets in approximately two seconds from the runner buckets to the energy absorbers in the walls of the wheel pit. Thus, speed rise will be held to a low value.

Low voltage leads. The six enclosed bus generator leads will be connected to metal clad air blast switchgear in the powerhouse. From the switchgear, three enclosed bus leads will carry the power up the shaft to the transformers. Air will be admitted into the enclosed bus ducts in the power house and released at the access building. Low voltage cables for this installation might have cost a little less but at nearly equal cost the enclosed bus was preferred.

High voltage equipment. Single phase 13,800/220,000 kv-37,500/50,000 kva transformers are furnished by Westinghouse. Their 50-ton shipping

weight is under the 60-ton hauling limit for the Haas site whereas two three-phase transformers of some 80 tons each could not be transported. Because the full plant capacity is dependent on any one of the single phase transformers and the plant can be snowed in, a spare is provided. At Balch, where access is better, three phase transformers of 68 tons shipping weight can be transported and no spare is provided. Oil-blast circuit breakers connect the circuit to a single circuit 220 kv transmission line.

Underground Power House

Dimensions. A basic design criterion was to put all facilities, valves, units, control room, tailrace gates, etc., in one excavation. It was considered that this would give lowest cost plant and the safest excavation. The height, crane rail to the top of stator, for installation of rotor was carefully determined to be a minimum. Factors that make this dimension a minimum are: (1) use of two bridge cranes with equalizing beam, (2) connection of hooks to equalizing beam to permit top of beam to raise to bottom of crane bridge, (3) lifting thrust ring on shaft to be as far down on shaft as possible, (4) flange between generator shaft and turbine shaft to be as close to rotor as possible, and (5) lift the shaft flange only enough to clear top of stator. At Haas only six inches clear over stator coil was allowed after all dimensions were determined. An excavation width of 56 feet was governed by the need for passing the low voltage leads along the upstream side of Unit No. 1 and space for runner and tailrace access hatch downstream from the unit. Space for the turbine valves and needle removal does not govern but is just adequate without special blockouts in the walls. Crane access to turbine valves is not considered a requirement in determining power house width, since it seldom, if ever, is necessary after installation. At Haas it is conveniently provided to the Unit No. 2 valve because of the space required for the enclosed bus. The Unit No. 1 valve can be raised to basement floor by lowering the crane cable between bus bars, Fig. 7, and skidding it to the runner hatch for removal to main floor. The length of the excavation is based on minimum passage past Unit No. 2, turbine foundation block, 15 ft. clearance between nozzle servomotors of Units No. 1 and 2 to permit access between the units during construction, Unit No. 1 turbine block, rotor assembly space, and all remaining facilities crowded in four stories at the access shaft end of the chamber.

Arrangement. Fig. 6 illustrates the arrangement. An unusual feature is the discharge and tailrace gate arrangement. The runner hatches are as close as physically possible to the units. The runner will be lowered by a portable hydraulic device onto a small rail car. The car is bolted to the runner and the car and runner are lifted together through hatches of minimum size to clear the runner and car when they are hanging on edge. There is no room for a lift tailrace gate and hydraulically operated hinged gates, which also serve to improve tailrace hydraulics, are used. Power house compressed air, 125 psi, is used to operate the gates.

Rock anchor and gunite arch. It was considered that, other than for a few rock anchors, no actual support would be required. However, spalling in the arch could be expected and it was decided arbitrarily to secure the ceiling for personnel and equipment safety. A rock anchored and gunited arch was estimated to cost one-half as much as a concrete arch and have other

advantages. It can be accomplished with minimum delay and cost to the excavation and the rough surface is desirable accoustically. The anchor-gunite arch requires a shorter span excavation and less volume of excavation. Details of the arch are shown in Fig. 6. The rock anchors are 1 inch diameter reinforcing bars, 10.0, 12.5 and 15 ft. long, and at 3-1/2 foot spacing both ways. The anchors and gunite in effect provide a reinforced or sewn rock arch, from which a loosened rock cannot fall. The Perfo Method of the Sika Chemical Corporation was selected as the method for overhead grouting. Two half round perforated tubes of sheet metal of the full length of hole are filled with a stiff sand cement grout and then wired together. This is inserted in the hole and the reinforcing rod is driven in by an air hammer. The volume of grout is such that a slight amount is extruded as the rod is driven to the end of the hole. 4" x 4" x #6 x #6 gunite mesh is formed to cover the rock with 1-1/2 in. minimum clearance over rock. Two #4 by 2' -0" reinforcing rods were welded to the anchor at 90° to each other and each parallel to the rock face. These bars supplement the mesh and anchor the gunite to the rock. Gunite of 4 in. thickness, and 1-1/2 in. minimum cover over mesh and rock points, was applied to the shape of the rock.

Formation drainage. Damp areas will be drained and not grouted. Three-inch diamond drill drain holes about 50 feet long and sloping upward are to be drilled into wet or damp zones in the power house chamber walls. Cores will be obtained to give knowledge of the rock near the chamber walls.

Cranes. Two 135-ton bridge cranes that require no alterations are to be taken from P. G. and E.'s Rock Creek power house on the Feather River. One of the two 110-ton cranes from the nearby Cresta power house will be moved to Rock Creek. The rotor lift is very seldom made after erection. If it ever becomes necessary to do major maintenance to the generator, it can be done in place or the rotor can be jacked up using the equalizing beam supported on cribs outside of the stator. If cranes were to be purchased for Haas, two rather than one would have been used. The cost of two cranes themselves is greater than the cost of a single larger crane but when structural steel and additional height of building is taken into account the overall cost of two as against one is about the same. Installation of the cranes is made very early and the construction advantage of two cranes is valuable. Two cranes give minimum height and excavation for the underground power house. The use of equalizing beam permitting the top of the shaft to extend up and between the cranes and the reduction of distance above crane rail to house the trolley reduces the power house height. Another example of economy in cranes is the use of a temporary construction crane for rotor installation and a permanent 25-ton gantry for the Company's outdoor Feather River plants.

Structural Steel. The structural steel crane columns and girders, Fig. 6, are installed immediately after the excavation is completed in order to make the cranes available for construction. The steel crane columns all go down to the rock excavation. Columns are 14 WF 136 with web parallel to the crane girders. The web of the 58-foot column is tied to a 1-1/2-inch diameter rock bolt every 14 feet for 1/b support. The 36 WF 194 girders are centered on the columns and the upper flange is rock-bolted to the walls to take lateral loads. The steel is shop sand blasted and shop coated with red lead. The walls will remain as exposed rock. The multistory construction

above main floor for control room, low voltage switching and power plant auxiliary services is a structural steel frame with reinforced concrete floor, concrete block walls and Robertson decking roof. This structural steel also goes down to rock so that it can be erected with the crane steel. It thus permits early installation and use of the elevator and stairs and provides early floor space for construction use.

Concrete. 3000 psi concrete and 1-1/2-inch maximum size aggregate is specified for the power house and machine foundations. Construction joints are arranged to give three basic pours. The first pour includes all possible concrete that can be poured before installing the turbines. This includes wheel pits, sub-basement floor and all the concrete at the service end of the power house. The main floor can be poured up to Unit No. 1. This first pour is to provide early construction space and access from the shaft and elevator. The second pour is the turbine generator blocks along with adjacent basement and main floors except for most of the area between units which is essential for access of several trucks under the cranes. The third pour is between units and will be made as late as possible since it will limit tailrace access to one vehicle under the tailrace hatch and under the power house cranes.

Ventilation. Air is drawn from each side of the tailrace into two independent and identical systems, Fig. 7. Each can handle 15,000 CFM and consists of a cooler-dehumidifier, blower and heater in the sub-basement. Cooling and dehumidifying coils use the cold tailrace water as the cooling media. Heating is by electricity (160 kw total). Cool air is delivered by 36 in. x 36 in. ducts to the crane rail level at each corner of the power house where it is sprayed into the chamber over each unit. The only other release is a small 2000 CFS release to each end of the sub-basement. Between the dehumidifier and blower is a return air damper which is automatically adjusted with the fresh air damper when heat is required. The bottom of the access shaft is air sealed from the powerhouse and air tight elevator doors are used in the power house. Outgoing air is admitted from the basement and main floors to the shaft; or upon temperature rise at the top of the chamber, directly to the access shaft through a motor controlled damper. The outgoing air goes up the unlined rock shaft and not in a duct. It will be beneficial in keeping the shaft dry and minimize blower requirements and cost of ducts. A 22,000 CFM blower is mounted at the top of the access shaft to aid in exhausting air from the power house. Two separate small systems are also provided. One is a quality air conditioning unit for the control and adjacent second story rooms. The other is two 8000 CFM blowers to the bus ducts which discharge at the roof of the access building and controls temperature rise in the ducts. Either blower may be the preferred unit with the other as standby. Battery room air is exhausted through a transite pipe to the shaft.

Lighting. Machine hall lighting will be by Hi-bay fixtures suspended from arched ceiling with supplementary light from wall bracket fixtures mounted on the crane columns. The vertical panel lights near the main floor level on each crane column will increase intensity and reduce shadows near the floor level. The balance of the plant, except for control room will have incandescent fixtures. The quality lighting is in the control room where a uniform level of illumination on the vertical surface of the switchboard and high wall

reflectance is desired. "Area-Liter" large square modul type fixtures were used with plastic instead of glass bottom and with rapid start lamps for higher lumen output and lower power consumption. A two-section, hollow square pattern is used to accommodate air vent diffusers, maintain distance from switchboard and proximity to walls.

Power House Access

Access shaft. The access shaft, Fig. 6, is 389 feet long and 18 feet in diameter. The location of the shaft at the centerline of the service and control end of the power house chamber is to give minimum length of shaft, minimum length of 13.8 kv bus, and elevator access to all floor levels, One of the exploratory drill holes was at the shaft and, except for the upper 20 feet, the cores were perfect. The circular shape rather than a more efficient square shaft was used to simplify excavation and to give the most favorable shape for an unlined shaft. The shaft contains all services between the underground chamber and the surface: 13.8 kv bus, 2.5 ton elevator, stairway, control leads, air and water pipes, and serves as the exhaust air duct. Platforms of 8 I 18.4 members at every 60 feet are securely anchored to rock. Intermediate platforms at 12-foot spacing are supported by hangers and then rock dowelled and welded to give lateral rigidity. The method of hangers was adopted to speed erection. All metal in the shaft is galvanized.

Elevator. The 2.5-ton capacity is governed more by construction than by operating requirements. The car is 12 feet high by 5' -10" x 4' -6" in plan. It can handle the generator pole pieces. The 12-foot height will permit two-level floor space for construction personnel. Specifications called for a geared traction, 300 ft. per minute, automatic self-leveling elevator. Landings are at basement, main, and second floor levels in the power house and at 60-foot intervals in the shaft. The elevator is furnished and installed complete with guide rails and doors by Otis Elevator Company. It is installed along with the structural steel in order to be available during construction.

Access building. A concrete block building with steel beams and Robertson decking room, Fig. 4, is located on top of the shaft. The second story houses elevator equipment, communications room and storage. Communication room equipment includes a radio to receive daily water level readings from the unattended Courtright and Wishon storage reservoirs. The elevator hoist is mounted on a rigid structural steel frame. Beneath the second story is the elevator lobby, change room and tool room. The single story portion of the building is a service and garage area, 42 x 36 feet in plan. An overhead 2-1/2 ton monorail crane can unload trucks to the elevator dolly. There can be several feet of snow at this site and some covered space for working, garage, and warehouse is necessary.

Tailrace tunnel. The railrace tunnel, Fig. 4, is slightly larger in width and height than the economic size for hydraulics, in order to provide the additional services of main construction access, heavy maintenance access, emergency personnel access and ventilation. The tunnel grade and the unit setting were lowered to take advantage of the hydraulics of the wider tunnel and concrete lined invert.

Tailrace Setting and Afterbay

For a Francis turbine, a setting involving no loss of head between plants is possible, but for the impulse turbine some head is lost. The design setting shown on Fig. 4 loses very little head. For full load operation and reservoir level at elevation 4097, the water surface at upstream end of the tailrace tunnel will be at elevation 4100, 6 feet below nozzle centerline. In the wheelpits, the surface of the aerated water may be about 4 feet below nozzle centerline. The units should operate normally with 2 or 3 feet of overpour at the dam. Greater overpour would be infrequent due to the upstream storage reservoirs and to the sluice capacity through the Balch Diversion Dam. The setting is practically the same as if the afterbay were at a controlled level. The surge in the tailrace tunnel from the one minute load-on time does not control the setting. Provisions have been made to test the operation of both Haas and Balch vertical shaft impulse turbines with compressed air in the wheelpit when tailwater is high. It will be of particular value to establish how much air is required and whether such operation is practicable.

The Balch Diversion Dam is a concrete arch dam with an overpour crest for the full 400 feet crest length. It will pass the design flood with a depth of 10 feet which would submerge the runner. The sluice capacity is 1700 cfs and is provided to limit overpour that would restrict Haas operation, and to sluice silt from the reservoir. Crest gates on the dam were considered but not used because of their high cost compared to the adopted design. Excellent abutment granite permits the overpour crest from abutment to abutment. Some rock anchors and abutment paving will be required.

The Balch Diversion Dam, Fig. 11, was constructed to its initial height in 1957 and is now being raised to the ultimate height. Two construction seasons are scheduled for several reasons. In 1957 a 60-inch diameter sluice is being installed and the existing dam thickened. This will give the new concrete time to reach equilibrium temperature with the old concrete before raising the dam in the late summer and fall of the following year. The low flow construction season is only three months. The thickening will be done with the reservoir empty and all work on Balch Project that requires shutdown of the small existing plant, Table of Fig. 1, will be done in 1957. The existing unit will not be shut down in 1958. Completion of the dam will be at about the same time as the power house so that a cofferdam for the Haas tailrace tunnel portal is not necessary. The design and construction of the raising of Balch Diversion Dam, other than it affects Haas Project, as discussed above, is not within the scope of this paper.

Project Construction

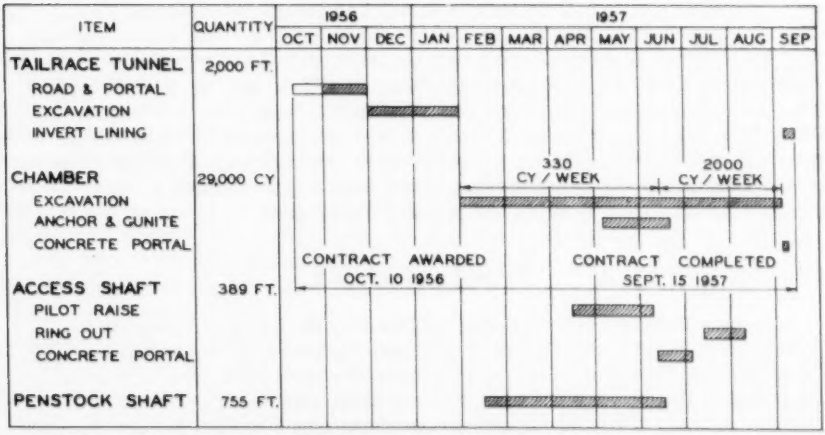
The Haas Project work has been awarded on the basis of competitive bidding in six contracts. For each contract, the name of the contractor, the schedule, and the statue of work in October 1957 are presented in Fig. 8. Courtright Dam, storage dam for all the plants, and Balch Diversion Dam, afterbay dam for the Haas power house, are included in Fig. 8. The six contracts are due to the practical matter of scheduling the engineering, construction, and expenditures, rather than a preference for smaller contracts. Haas and Balch penstocks were bid together and also Haas and Balch power houses. Thus contractors for these jobs are doing work at the same time on the Balch Project. It is observed from Fig. 9 that the contracts, except Wishon

*CONTRACT	CONTRACTOR	START	COMPLETE	**MONTHS	% COMPLETE OCT. 1957	WORK SEASON (MONTHS)
Courtright Dam	Morrison-Walsh-Perini	5/56	11/59	43 (20)	55	5
Wishon Dam	Morrison-Walsh-Perini	5/55	11/58	43 (28)	80	7
Wishon Reservoir Clearing	Dreisbach Box & Lumber Company	5/56	11/57	19 (14)	98	7
Haas Tunnel	Morrison-Kaiser-Macco-Perini	10/56	10/58	24	65	12
Haas Penstock	Consolidated Western Steel Company	4/57	11/58	20	20	12
Haas Powerhouse Excavation	Morrison-Kaiser-Macco-Perini	10/56	9/57	11	100	12
Haas Powerhouse Civil	Rothschild-Raffin-Weirick	10/56	12/58	11	0	12
Haas Powerhouse Elect.-Mech.		11/57	12/58	11	0	12
Balch Diversion Dam	Intrusion Prepakt	8/57	11/58	16 (6)	10	3

* Courtright Dam and Balch Diversion Dam are not directly a part of the Haas Project. Courtright Reservoir provides storage for Haas, Balch and Kings Projects. Balch Diversion Reservoir is Haas Afterbay Reservoir.

**Months in parenthesis are working months.

FIG. 8 HAAS PROJECT - CONTRACT AND SCHEDULE DATA



NOTE: WORK WEEK IS 3 SHIFTS PER DAY AND 6 DAY WEEK.

FIG. 9 HAAS POWER PLANT - EXCAVATION PROGRESS

clearing and Haas power house excavation, are completed at the same time though they vary in length from one to four years. This, of course, is normal practice to keep interest during construction to a minimum. The closeness of the schedule in the last months of construction presents many crises.

Major equipment such as penstocks, valves, structural steel, turbines, generators, and transformers are ordered and purchased by the Company and furnished to the contractor for installation. All such items are usually on order in advance of awarding the construction contracts. Smaller items, where delivery is not a problem, are furnished and installed by the Contractor. Purchasing and administration of contracts is by the Company's Purchasing Department, and supervision, and inspection of the work is by the Company's Construction Department.

Dam and Conduit Construction

Construction of the two rockfill dams and of the power tunnel is well along and ahead of schedule. Details of methods and equipment are well covered and illustrated in the engineering construction literature[3,6,9,10,13,14] and will be summarized and supplemented in this paper. Figs. 12, 13 and 14 show rockfill dam construction for Courtright Dam, construction being the same as for Wishon. Power tunnel[10] and surface penstock construction will be reviewed briefly.

Wishon Dam

Quarrying. About 2,500,000 cubic yards of solid rock, 3,700,000 cubic yards or rockfill, are required for the dam. The dam is surrounded by exposed massive granite. The Specifications required that the spillway be one quarry and that one or more others, not closer than 200 feet to the dam cutoff, be located to suit the contractor. The spillway floor is made essentially level and wide in order that it be a satisfactory quarry. No payment is made for spillway excavation, except as rockfill in the dam. Rockfill is paid for by a specified price of 50 cents a cubic yard solid in the quarry plus a bid price per cubic yard in the dam. The 50 cents is not intended to fully cover quarry costs. The pricing method is such that both the contractor and owner lose money when material is wasted, which reduces risk to the contractor and contingency money in his bid. The specified spillway quarry may be enlarged on the side away from the dam to suit the contractor. An enlarged spillway quarry on the left abutment and a major quarry near the right abutment are being used. The right abutment quarry was developed in two main benches, each 50 ft. high.

Dumped rockfill. The rockfill is sound, hard, and durable granite. The Specifications require, "At least 50% by weight of all the rock in the entire dam shall range from 1/2 to 10 tons and over. . . rock handling equipment shall be of ample capacity and strength to load, transport, and place individual rock to at least 15 tons in weight." The objective is to strive toward clean large quarried rock and be able to handle it. Quarry run rock with essentially no waste is being used. Main lifts are higher than the specified 60 foot minimum. Sluicing is specified to be at not less than three times the volume of rock being placed and by nozzles of 2-inch minimum size. Two and one-half or three inch nozzles are being used. The pumping system design is

based on 100 psi minimum pressure at the nozzles, but a minimum pressure of 70 psi is permitted. Peak production is at a rate of 16,000 to 18,000 cubic yards per day.

Placed rock. The design and specifications, and the contractor's well planned production operation, combine to increase the speed and lower the cost of the placed rock.(9,10,13,14) Peak production of Lower Bear River dam averaged 45 cu. yd. per crane shift for one month of 670 crane shifts. At Wishon the rate peaks at about 200 cu. yd. per crane shift and averages 135. Design changes since the Bear River Dam that speed up the placing are: (a) use of face lifts, (b) elimination of keyways, and (c) easier placed rock specification. Face lifts are small lifts about 30 feet high with 40 foot road width on top. The upstream face of the main lifts is kept back about 40 feet to leave space for the face lifts which provide access to the placed rock and concrete, Fig. 14. The placed rock is of large size, 5 to 15 tons, on both dams, Fig. 13.

Concrete face. The concrete face is placed by buckets handled from a crane on the face lift. A slip form pulls itself up the face by electric operated winches on the slip form. It can be moved by cranes operating on the face lift. The face lift and slip form combination make a straightforward job. Aggregate is produced from pit run material located in the reservoir site. The presence of many cobbles and little 3/4 to 1-1/2 inch material makes crushing necessary. Because of the friable nature of the aggregate, it is furnished in four rather than three sizes and rescreened at the batch plant to assure grading that will result in a dense frost-resistant concrete. The 3000 psi air entrained concrete contains Pozzolith, pozzolan and Type II cement.

Power Tunnel

The tunnel is being driven from one adit and on two headings. The 32,850 ft. length is divided into a 19,560 ft. upstream leg and a 12,830 ft. downstream leg. The first 2800 ft. in each direction from the adit was driven economically by alternate headings at an average rate of 90 feet per day, for one crew on each of 3 shifts, until daily progress dropped to the required schedule of 1000 feet per month (40 feet per day) in the long upstream heading. To October 1957, 21,000 feet have been driven at an average rate of 53 feet per day per heading. Maximum days are 70 to 75 feet per day per heading and recently monthly averages are 65 to 70 feet per day per heading. Of the 21,000 feet, only 413 feet, in several zones of faulted rock, were supported. Two soft seams, 3 to 6 in. wide, will be cleaned out to 6 in. depth and gunited. Five hundred feet of tunnel at the dam was included in the dam contract to avoid later damage to the intake works by the holing through. The intake works must be in operation a year before holing through to begin storing water in the reservoir. At the downstream portal about 150 feet of timber supported tunnel has been driven from the portal until good rock was encountered, in order to hole through in unsupported tunnel. Survey accuracy on holing through was 0.0 feet in grade and 0.1 feet in line.

Surge Chamber

The 300-foot long by 12-foot diameter shaft is being excavated from the top and bottom. A head frame has been installed. Drill holes indicated that

the upper 150 feet is in poor granite and it was considered safer to excavate down rather than up. It has been in fractured and soft grained granite and has been concrete lined. The procedure is to excavate 10 feet and then line it. The bottom 150 feet is being driven as a raise from a steel cage suspended from a cable in a 6-inch drill hole. A 4-inch gunite lining with 4 x 4 x #6 x #6 mesh will be used. The galleries at the base of the surge shaft, Fig. 4, will be unlined, gunited, or concrete lined, as rock conditions require.

Surface Penstock

About one-third of the way up the penstock a road crosses the penstock. An old gantry has been set across the road to transfer the 40-foot long and 12 to 40-ton sections from truck to the tram cars. Two separate cars with saddles, and connected by cable, make it possible to negotiate the vertical and horizontal curves. Below the road the tramway is on penstock centerline and installation will proceed from the bottom of the shaft to the road. Above the road the tram parallels the penstock and sections will be rolled and skidded over to the penstock line, and erection will begin at several anchors. The expansion joint travel is made such that the expansion joint below an anchor may be rolled in place and expanded into the line as a closure section. The first pours of piers and anchors are made in advance of pipe erection. In several zones, of several hundred feet each, the foundation material is undisturbed, decomposed granite. It is good foundation material but very subject to weathering and erosion. In such areas, the penstock trench and 4 feet up each side is protected by 2-inch gunite with 4 x 4 by #8 x #8 mesh, applied to the irregular excavated surface without trimming. Drainage is taken away from the penstock as frequently as the topography and penstock cuts will permit.

Underground Plant Excavation

General

The power plant excavation contract, Fig. 9, included tailrace tunnel, power house chamber, access shaft and penstock shaft. The contract was completed on September 15 and work on the Civil Contract began on the same day, which leaves one year and two months for power plant construction. All the excavation was uneventful in that practically no support was required and no water was encountered. Fig. 9 is a record of the excavation progress, and Fig. 10 illustrates the excavation methods. Details of the excavation has been well covered in the engineering literature(7,10,12,13) and will be summarized and supplemented in this paper.

The excavation contract included some concrete work considered essential to turn the job over to the next contractor in safe condition and with unrestricted access. A concrete collar of 18-foot inside diameter was installed at the top of the access shaft to a depth of 20 feet to cover the surface zone of fractured and weathered rock. A ten foot long portal, of tailrace tunnel inside dimensions, at the wall of the power house was installed as its later forming would interfere with access. The pouring of the tailrace tunnel invert was also included.

The first step in the excavation contract was to complete the tailrace tunnel in order to gain access to the power house and to the other excavations.

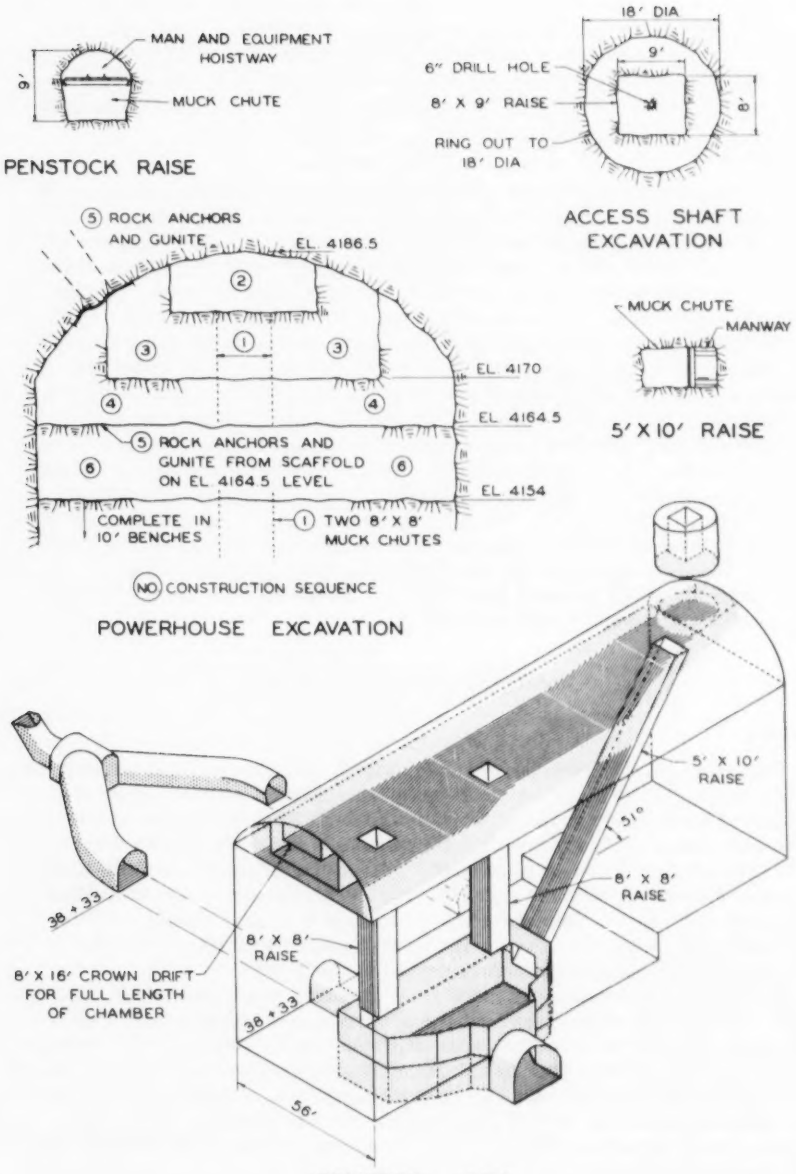
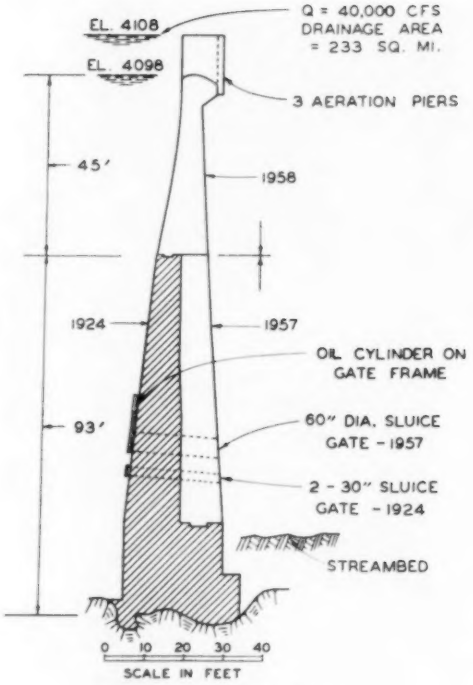


FIG. 10 HAAS POWERHOUSE EXCAVATION



MAXIMUM SECTION

FIG. 11 BALCH DIVERSION DAM

FIG. 12 COURTRIGHT DAM - DUMPING AND SLUICING ROCKFILL - 140 FOOT LIFT

FIG. 13 COURTRIGHT DAM - PLACED ROCK, JOINT RIBS, AND CONCRETE FACE SLAB.

FIG. 14 COURTRIGHT DAM - PLACED ROCK CRANES ARE ON FACE LIFT.

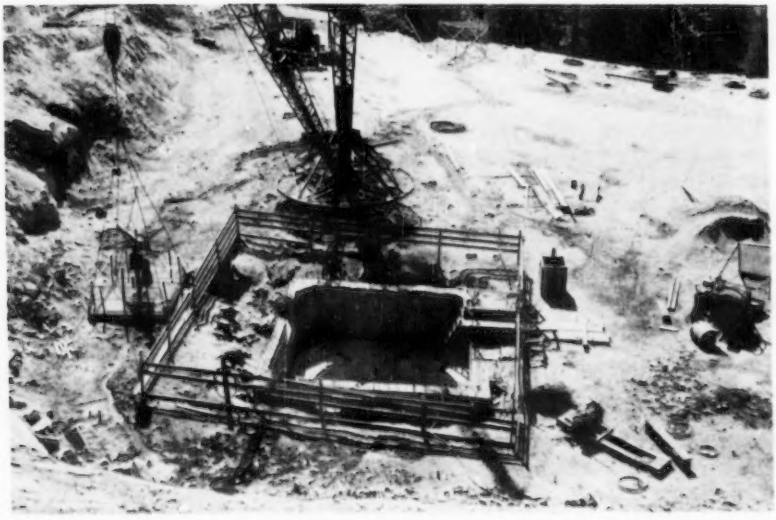


FIG. 15 HAAS POWERHOUSE - TOP OF ACCESS SHAFT AND BASEMENT OF ACCESS BUILDING.

FIG. 16 HAAS POWERHOUSE - GUNITE ROOF

A small chamber was excavated, from which work was begun simultaneously on the penstock tunnel and shaft, two 8 x 8 ft. raises to the crown of the chamber, and a 5 x 10 ft. 51° raise to the base of the access shaft, Fig. 10. As indicated by Fig. 9, the excavations proceeded together with the chamber excavation determining the completion date.

Tailrace Tunnel

Excavation. The 2000-foot long, 15' x 18' high, tailrace tunnel was driven in 58 working days at an average rate of 34 feet per working day which includes one and two shift days. For three shift driving, the average was 47 with maximum days of 55 to 60 feet per day. Thirteen foot rounds were pulled and mucking was by Eimco mucker and Dumptors.(6,12) Four 40-foot long passing areas were constructed, each giving a total width of 20 feet for passing instead of the tunnel's 15-foot width. No support was required and no water was encountered.

Invert lining. Upon completion of the chamber excavation, it was necessary to pour the 2000 ft. length of invert rapidly to give the power house contractor access to the job. The pour was made in 40 hours. The muck was removed down to rock points by equipment. A profile was taken and a grade established to give a 5-inch minimum slab that was at a variable slope between horizontal and design grade. This departure from design grade gave maximum hydraulic section and minimum concrete. The invert concrete is a plain slab with no gutters or curbs, Fig. 4.

Power House Chamber

The size of the initial working chamber, 25 x 70 feet in plan and 23 feet high, was governed by the wheel pit and sump area being at tailrace grade and by the top of the penstock tunnel, Figs. 7 and 10. Two 8 x 8 ft. raises were driven to the crown and an 8 x 20 ft. crown drift was driven the full 173 foot length of the power house. In order not to interfere with the access shaft excavation, the excavation of the arch, rock anchoring, and guniting (construction steps 3, 4 and 5 of Fig. 10) were started at the opposite end. The rock anchor and gunite work, step 5, was carried out from pipe scaffolding on floor elevation 4164.5. Scaffolding was moved as work progressed toward the access shaft end of the power house. The work was well organized and the 2000 anchors and 13,000 square feet of gunite was completed in six weeks along with some remaining step 3 and 5 excavation (Fig. 10). After the arch was completed, the excavation proceeded rapidly, Fig. 9, at an average of 2000 cu. yds. per six-day week with daily production varying between 200 and 1100 cu. yds. per day. The main excavation was carried out in irregular benches about 10 ft. or more in height. Muck was brought to the chutes by a slusher. The excavation between the two 8-foot square chutes was completed first in order not to interfere with the access shaft excavation. Only a few rock anchors were used other than the pattern, Fig. 6, arbitrarily established for the roof.

No water was encountered but there were several damp spots on one wall, Fig. 17, and at each end. The moist seams will be drilled to drain them and minimize dampness, but no curtain walls will be used. The rock temperature was 58° when the tailrace tunnel reached the chamber in February 1957 which is a very favorable temperature. It has increased about 0.5° per month due

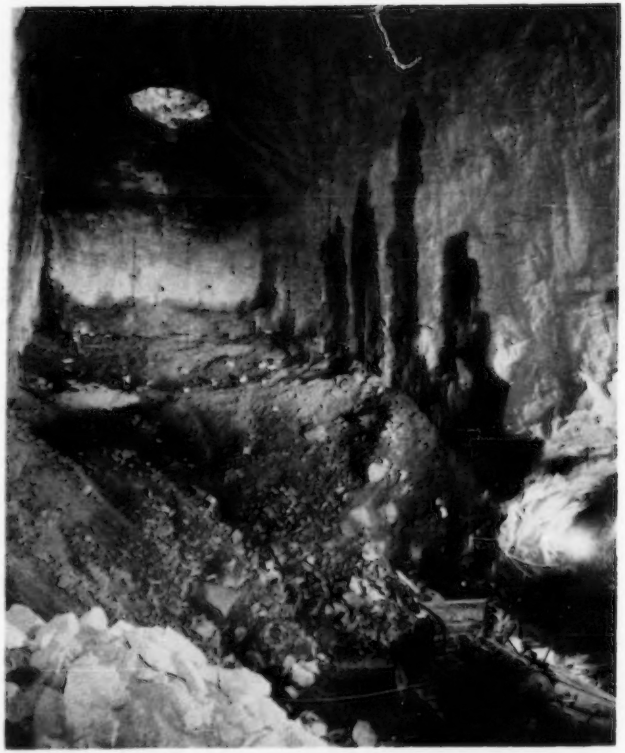


FIG.17 HAAS POWERHOUSE-CHAMBER EXCAVATION NEARING COMPLETION-DARK SPOTS ARE DAMP AREAS

FIG.18 HAAS POWERHOUSE- EXCAVATION COMPLETE STRUCTURAL STEEL AND CRANES INSTALLED

to the warm ventilating air, and in September 1957 was 62° F. Figure 18 shows the chamber with structural steel and cranes installed.

Access Shaft

The access shaft excavation, Fig. 10, began with a 6-inch diameter hole. An 8 x 9 ft. raise was driven using a cable supported steel cage. The cage was pulled under the chamber arch and the cable raised in the 6-inch hole when a round was to be shot. The 18-foot diameter was then ringed out from the top down. The top 20 feet of the shaft was in weathered rock and was excavated from the surface bench and concreted, Fig. 15, before ringing out the 18-foot unlined shaft, which is in excellent and dry rock. The concreted rock carried water and the formation was grouted. All the rest of the shaft will remain unlined.

The 389 feet of 8 x 9 ft. raise was driven in 25 working days at 16 feet per day. Progress increased from 8 to 15 feet initially to 20 to 25 feet per day near completion. Ringing out took 23 days at an average of 17 feet per day.

Penstock Shaft

After 635 feet of the 760-foot shaft was raised, fractures and then soft grained granite was encountered. The remaining 125 feet was driven from the surface bench downward and required timber support for safety. The poor rock extended lower than anticipated and the penstock design was changed to begin the increase in free pipe stresses where seamless granite began, Fig. 5.

Men and supplies were brought to the raise heading by a small car in the manway, Fig. 10. The car was pulled by a tugger hoist located in the 9-foot penstock branch excavation, Fig. 7. The muck was moved by rail through the 12-foot branch excavation and dumped into the chamber. The branch pipes are only 4 ft. -7-1/2 in. OD, but the 12 ft. size tunnel is necessary to install the two 82 in. diameter bend pieces and the wye.

The raise was driven in 113 days at an average rate of 7 feet per day with days varying from 5 to 10 feet depending on whether one or two rounds were pulled.

CONCLUSION

The appreciable economy in the use of rockfill dams and unlined tunnels has been obtained in a number of previous P. G. and E. projects. In the Haas Project, the rockfill dam design has used experience to provide a more simple and lower cost dam without changing its high degree of safety. The underground intake valve, gallery-simple-differential surge chamber and storage rock trap combine with the unlined tunnel to provide a simple and economic conduit. It is believed that the combined work of designers and contractors can further reduce the cost of concrete faced rockfill dams. In the case of unlined tunnels, further economics largely rest with the contractors.

Haas is the largest of three underground hydroelectric power houses in the United States.(8) In other two are Snoqualmie Falls, Washington (1899) and P. G. and E's Spaulding No. 1, California (1917). The combination of very high head, flat penstock profile near the river, and excellent rock

provided the conditions that made the Haas underground layout attractive. Since the major contingency item, excavation, has been completed without delay or mishap and other contracts have been awarded, it appears that the underground scheme at Haas is in fact, as well as in estimate, substantially more economic than the surface scheme.

ACKNOWLEDGMENTS

Engineering is under the direction of Walter Dreyer, vice president and chief engineer and C. W. Appleford and W. R. Johnson, chief civil and electrical engineers, respectively. Much of the project planning and engineering was handled by H. V. Lutge, chief civil engineer prior to July 1956. I. C. Steele is consulting engineer on the dams. The writer is indebted to J. E. Schumann, A. G. Stressburger, and others who have been engaged on the engineering studies and design for the Kings Development.

Construction is under the direction of A. J. Swank, vice president in charge of general construction, and H. W. Haberkorn, manager of hydroelectric construction. Joe Pirtz is project superintendent and George Thacher, project engineer.

The major civil engineering contracts of dam, tunnel, and power house excavation are sponsored by Morrison-Knudsen Company, represented by Jim Wells, vice president. Burt L. Perkins, Whitey Lee and Carl Larsen are project managers and John F. Erdle is project engineer.

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5. CALIFORNIA HYDRO PROJECT TO HAVE FIRST BIG UNDERGROUND PLANT IN U.S. Engineering News Record V. 157, Oct. 1956, pp 26-27.
6. TWO ROCKFILL DAMS ON KINGS RIVER Southwest Builder and Contractor, Oct. 12, 1956, pp 16-24.
7. HAAS UNDERGROUND POWERHOUSE (Another Phase of $80 Million Kings River Project). Pacific Road Builder and Engineering Review V. 88, June 1957, pp 25-33.

8. BIBLIOGRAPHY UNDERGROUND HYDROELECTRIC POWER PLANTS J. B. Cooke and A. G. Strassburger. Proceedings of the American Society of Civil Engineers, V. 83 No. PO4, Aug. 1957, p 1350-1 through 1350-36.

9. BUILDERS SAVE MILLIONS ON TWO FACED-ROCKFILL DAMS. L. L. Wise, Engineering News-Record V. 159, Aug. 29, 1957, pp 38-44.

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11. PENSTOCK EXPERIENCE AND DESIGN PRACTICE OF P. G. AND E. COMPANY G. V. Richards. Proceedings of the American Society of Civil Engineers V. 83 No. PO5, Oct. 1957.

12. P. G. AND E. GOES UNDERGROUND FOR POWER L. L. Wise. Engineering News-Record V. 159, Sept. 19, 1957, pp 51-.

13. EFFICIENT ROCK PLACING SPEEDS HIGH DAMS AND CREWS HOLLOW MOUNTAIN TO BUILD UNDERGROUND POWER HOUSE Construction Methods and Equipment, V. 39, Sept. 1957, pp 158-162 and 166-175.

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Note: Paper 1538 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 84, PO 1, February, 1958.

WATER HAMMER DESIGN CRITERIA[a]

Discussion by S. Logan Kerr and R. S. Quick
Closure by John Parmakian

S. LOGAN KERR,[1] M. ASCE.—It is extremely valuable to have a summary of design criteria, such as Mr. Parmakian has presented in his paper, before entering into a detailed investigation of water hammer on any power or pumping project.

In addition to the controlling features of design such as capacity, length of flow line and other factors, it is essential to investigate with great care the proposed methods of operation of the plant. In many instances the operating conditions can be modified and substantial savings effected if rules of procedure are set down along with the design criteria.

In many cases the establishment of design and operation conditions at the outset of a project can simplify the water hammer studies and eliminate the need for the analysis of a large number of alternative calculations.

One factor in operating procedure on hydroelectric plants which should be emphasized is the degree of speed regulation required. In the early hydroelectric plants it was almost always the rule that the turbine governors were required to regulate the system speed without interconnections.

With the increase in transmission networks it is now the rare instance where precise speed regulation by the hydroelectric unit is required. The governor is thus relegated to the position of an overspeed limiting device, but even this requirement is unnecessary if the turbine and generator can withstand full runaway speed following a tripout of the station.

Some plants have been installed with no speed regulation and have been quite successful in operation. Under present day conditions in the United States, the term "speed regulation" and "hydraulic turbine governing" are secondary to frequency control. This eliminates the necessity for much of the high speed governor action. Water hammer resulting from a tripout of a hydroelectric unit may depend only on the change in turbine discharge as a function of speed.

With respect to motor driven pumping units, particularly those equipped with automatic check valves, there are a number of problems which arise and for which design criteria may be more complex and may require a more thorough investigation of water hammer conditions.

The basic list which Mr. Parmakian has presented in his paper is most helpful and should serve to clarify much of the discussions with regard to water hammer.

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a. Proc. Paper 1216, April, 1957, by John Parmakian

1. Cons. Engr., S. Logan Kerr & Co., Inc., Engrs., Flourtown, Pa.

R. S. QUICK.[1]—Mr. Parmakian has outlined how penstock design stresses can be established on a most economical basis consistent with safety, after making appropriate allowance for water hammer surge. The magnitude of these surges is related intimately to the timing of the governor and penstock valves and to the flow thru the turbine runner or pump impeller with varying head and speed, as the case may be. Accordingly, a clear understanding of these characteristics and the requirements of the installation will aid materially in establishing a proper range of operating condition.

In the case of turbines with governor operated gates controlling the flow, the required governor stroke timing should be as slow as operating requirements will permit. In large power networks, speed regulation as contributed by a single hydroelectric generating unit, is relatively unimportant, so the governor can be timed to limit overspeed on line separation, resulting in minimum pressure surge.

When the pressure rise exceeds about thirty percent it may be found more economical to employ a surge limiting device rather than to increase the strength of the penstock. Such a device can be a surge tank if both rejected and applied loads are to be handled, or a governor operated water saving relief valve if only rejected loads are involved. Relief valves are preferable under high heads or in locations where a surge tank is difficult or uneconomical to apply, Hoover Dam powerhouse being a good example.

An installation with a high pressure rise and no surge tank may be difficult to regulate. A relief valve is very effective in stabilizing such plants and should be considered where occasional isolated operation is required.

Under emergency conditions, penstock valves in hydroelectric units may be closed by excess flow actuated automatic devices or by push button operation from a local or remote point. Such valves are large relative to the flow and do not throttle effectively until nearly closed. Thus such valves should be timed with due regard to the pressure rise which they may create. Hydraulically operated valves can be cushioned to retard final closure while electric motor operated valves would require variable speed motors, a built-in variable rate mechanism, or a slow rate for the entire stroke.

When upper penstock valves are closed against flowing water, a generous air inlet system is required immediately downstream to prevent collapse of the downstream pipe as it empties and is subjected to a partial vacuum.

Discharge pipelines of electric motor driven centrifugal pumping plants are subjected to critical waterhammer, as a rule, on power failure. Mr. Parmakian's text "Waterhammer Analysis"[2] provides information on methods of determining such surges. When simple valve systems do not limit the surges to values permitting an economical penstock design, surge tanks, air chambers, variable rate check valves or surge suppressors may be found very effective. There is an excellent reference list of articles on this subject on pages 143, 144 and 145 of Mr. Parmakian's text, mentioned above.

High points on pump discharge penstock profiles can be protected from collapse due to a sudden down surge, and to excessive high pressure on rejoining of the water column, by use of fast opening and slow closing air valves. The design should include a dashpot timed closing mechanism so arranged as to insure discharge of the air in the pipeline followed by a slowly retarded flow of water as the normal hydraulic gradient is restored. In cases where

1. Cons. Engr., Eddystone Div., Baldwin-Lima-Hamilton Corp., Philadelphia, Pa.
2. Prentice-Hall, Inc. New York, 1955.

the gradient is near the static level, an open standpipe may suffice. In some cases, where the pumping gradient is above the static level by a substantial margin, a means to close the outlet of the standpipe while pumping, and to leave it open at other times, has been found to be very effective.

In the examples cited by Mr. Parmakian, the maximum normal and emergency pressures are determined for design purposes. However, there does not appear to be a specific mention of the maximum pressures which might occur under extreme emergency where the factor of safety of one would apply. A further explanation of this hypothetical situation would be helpful.

JOHN PARMAKIAN,[1] M. ASCE.—The author wishes to thank Messrs. Donsky, Kerr, Quick and Withers for their discussions. Mr. Quick and Mr. Withers requested a further explanation of the items listed in the design criteria as emergency conditions of turbine and pump operation which are not normally considered in the design. In general, it is not economically feasible to spend any extra money on an installation to accommodate these most unlikely emergency conditions of operation. However, it is somewhat assuring to a designer to ascertain if the likely factor of safety under these conditions is still above one. Ordinarily, low head pump or turbine installations will not be able to satisfy this condition as well as high head installations. For example, at the relatively low head Boysen Powerplant cited in the paper, the penstock near the turbines should theoretically be able to withstand a head of 732 feet with a factor of safety of one. However, the computed maximum head under the extreme emergency conditions is 985 feet. At the higher head Yakima Pumping Plant the discharge line near the pumps should theoretically be able to withstand a head of 1,344 feet with a factor of safety of one. The computed maximum head under the extreme emergency conditions is 939 feet.

Mr. Kerr and Mr. Quick mention the minor roll that the turbine governor sometimes plays when the unit is tied into a large interconnected system and is not required to regulate frequency. In general such operation will permit a slower governor action and hence reduce the pressure surges on the penstock system. However, as noted by Mr. Kerr, the reduction in the turbine discharge due to the higher overspeed of the unit may be the controlling factor in determining the water-hammer effects.

Mr. Quick mentioned the arrangement where "In some cases, where the pumping gradient is above the static level by a substantial margin, a means to close the outlet of the standpipe while pumping, and to leave it open at other times, has been found to be effective." This type of arrangement is described in Paper No. 57-A-25 of the American Society of Mechanical Engineers by the author entitled "One-Way Surge Tanks for Pumping Plants."

Mr. Donsky describes a typical use of surge suppressors and slow closing air valves in a pump discharge line where water column separation could not be avoided. As noted by Mr. Quick, the primary function of the fast opening and slow closing air valves is that of an automatic surge suppressor at the points of water column separation.

The writer agrees with Mr. Wither's statement that in the past some designers have included the water-hammer effects with other factors of

1. Head Technical Eng. Analysis Section, Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

ignorance concerning the capabilities of the pipe, and as a result, the overall factor of safety is unknown. With the present state of knowledge, water-hammer effects can be computed with a good degree of accuracy.

PENSTOCK DESIGN AND CONSTRUCTION[a]

Closure by G. R. Latham

G. R. LATHAM,[1] M. ASCE.—Mr. Hobson's discussion and drawings outlining the practice of the Army Corps of Engineers' design of penstocks is interesting and informative. As he states, the engineer's problem is to design each installation to suit the particular conditions and provide adequate construction at reasonable cost. The installations given are of large size ranging from 15 to 25 ft. diameter. While ring girders have been used for supports in a number of installations it is of note that at Garrison a considerable length of the 24 ft. diameter penstock is supported on 180° concrete saddles. This agrees with our investigations that concrete saddles are often more economical than large diameter field assembled ring girders.

As Mr. Heidinger states, there are no acknowledged standards for penstock design and many different procedures are used. The ASME Code followed by the Bureau of Reclamation requires a basic unit stress of 13,750 lb. for ASTM A-285 Grade C steel which has an elastic limit of 30,000 lb./sq. in. and an ultimate strength of 55 to 65,000 lb./sq. in. We believe this working stress very conservative, as compared with the allowable working stresses used in building construction where safety of the structure is an important consideration. The AISC permits a working stress of 20,000 lb./sq. in. for ASTM A-7 steel which has an elastic limit of 33,000 lb./sq. in. and an ultimate of 60 to 70,000 lb./sq. in. The AWWA and the API Codes both allow more liberal working stresses than the ASME Code.

A pipe is basically a simple structure efficiently using metal to withstand the principal stress from internal pressure. There are also important localized stresses caused by such conditions as filling and emptying, beam action between supports, saddle and ring girder reactions, temperature changes, vacuum and for buried pipes external pressure. Most of these conditions are temporary in nature and the probability of their occurring simultaneously is small. The choice of allowable unit stress for such conditions is a matter of engineering judgment; we consider that 18,000 lb./sq. in. is a reasonable basic unit stress which provides a reasonable margin of safety. We have been guided by the various codes and our previous experience on constructing many satisfactory installations over a long period of time. Control of welding is under qualified inspectors to insure that it conforms with good practice.

We have constructed many riveted penstocks up to 18 ft. diameter using a 16,000 lb./sq. in. unit stress. These were filled under static head and examined for leaks. They have given very satisfactory service and are

a. Proc. Paper 1285, June, 1957, by G. R. Latham.

1. Chf. Architectural-Structural Engr., Ebasco Services, Inc., New York, N. Y.

continuing to do so. When the quality of steel improved in the Twenties the allowable stress was increased to 18,000 lb./sq. in. When welding procedures improved and welding came into general use it was adopted for penstock fabrication. The initial installations consisted of shop welded sections about 30 ft. long with field riveted circumferential joints, design being based on 18,000 lb./sq. in. unit stress. As the art of welding developed and good field welders became available the circumferential joints were also field welded. These installations have been satisfactory and our experience has indicated no necessity for a costly and often difficult overpressure test of the completed penstock. However, the writer would like to mention that the shell plates of these field welded penstocks that we have constructed have not exceeded 1 in. thickness.

For the Kerr HED, the penstock shell was designed for the full head including water hammer with no load considered to be taken by the encasing rock. It is appreciated that the steel shell will yield under load and transmit some pressure to the surrounding concrete and rock. The design thickness of 15/16 inch was considered rigid enough for handling without the use of stiffeners but if the thickness was reduced by permitting the rock to take part of the load, the thinner plate would have required stiffening for handling. In view of the short length of the penstock (approximately 100 ft.), it was decided not to attempt to utilize the rock in resisting the pressure.

The Pelton penstocks are supported on and buried under a controlled compacted backfill. Medium to fine sand was used for backfill, compacted with a vibrator roller, resulting in a very tight, hard compacted fill. In addition to the 16 ft. backfill load on the penstocks, there is a superimposed transformer load in certain portions. The vertical load was considered to act uniformly across the diameter, and the horizontal soil pressure was taken as one-half the vertical load also acting uniformly across the diameter. The stiffeners were designed for the backfill loading plus the water load due to a "just-full" condition.

The papers and discussions resulting from the "Penstock Symposium" of the Buffalo meeting have been very interesting and illustrate some of the possible design variations. The interchange of information on design methods and the results experienced are of value to engineers responsible for designing safe installations at reasonable cost.

BIBLIOGRAPHY: UNDERGROUND HYDROELECTRIC POWER PLANTS[a]

Discussion by George A. Whetstone and Stavros N. Nicolaou

GEORGE A. WHETSTONE,[1] M. ASCE.—The authors have made a painstaking search in assembling the list of underground plants and the 213 bibliographic references. They are to be congratulated on their thoroughness.

Whether or not the plant at Snoqualmie Falls should be regarded as the first underground plant depends on a technical definition; while it seems to have been the earliest installation in which the sole purpose of the excavation was to provide a powerhouse, examples exist of hydroelectric power plants installed deep in mines as early as 1890. Those of the Comstock Lode with heads of 1680 feet (1890) and 2100 feet (1892) were among the foremost hydraulic achievements of the nineteenth century.

The following additions to the bibliography are submitted in the interest of completeness.

1890

THE ELECTRICAL TRANSMISSION OF POWER. Engineering News v 23 p 341. Tailrace from Nevada Mill surface power plant carried down vertical mine shaft by two iron pipes to supply six Pelton wheels under a 1680-foot head, 125 hp each. "Greatest head of water ever used by any wheel."

1892

PELTON WHEELS UNDER 2100-FOOT HEAD. Engineering News v 27 p 538, 610. One wheel, 100 hp, with 460 feet of head from the pipeline of the Gold Hill Water Co. plus 1640 feet from the mine shaft.

1899

THE SNOQUALMIE FALLS ELECTRIC POWER PLANT. Engineering v 68 p 420-422. Primarily interested in electrical details.

1900

THE SNOQUALMIE FALLS WATER-POWER PLANT AND TRANSMISSION SYSTEM. Engineering News v 44 p 398-404 + plate.

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a. Proc. Paper 1350, August, 1957, by J. Barry Cooke and Arthur G. Strassburger.

1. Prof. of Civ. Eng., Texas Technological College, Lubbock, Tex.

1901

INSTALLATION HYDROÉLECTRIQUE DE SNOQUALMIE FALLS. Le Génie Civil v 38 p 317. Description and sketch.

1914

L'USINE HYDROÉLECTRIQUE DE MOCKFJORD (SUÈDE). Le Génie Civil v 64 p 298. Description and cross-sectional view.

1916

L'USINE HYDROÉLECTRIQUE DE PORJUS. Le Génie Civil v 68 p 72-76. Detailed description with 11 figures.

1931

LES USINES HYDROÉLECTRIQUES DE BROMMAT ET DE SARRANS DE LA SOCIÉTÉ DES FORCES MOTRICES DE LA TRUYÈRE. Ch. Duval. Revue Générale de l'Électricité v 30 p 997-1011, 1027-1042. Profusely illustrated description of installations.

1947

L'AMÉNAGEMENT DE L'ESTUAIRE DE LA RANCE ET LA CRÉATION D'USINES MARÉMOTRICES SOUTERRAINES. L. Garoult Le Génie Civil v 125 p 198 (Abst from Le Monde Souterraine, Dec. 1947). Suggests a vehicular tunnel and underground tidal power plants in a scheme for developing the Rance.

1950

L'AMÉNAGEMENT HYDROÉLECTRIQUE DE L'OUED AGRIOUN (ALGÉRIE). R. L. Bibard. Le Génie Civil v 127 p 398 (Abst. from Industrie des Voies Ferrees et des Transports Automobiles, Juil-Août 1950). Outline of projected development with numerical data.

L'ENERGIE HYDROÉLECTRIQUE AU PÉROU ET AU CHILI. Marc Pieyre. Revue Générale d'Hydraulique v 16 p 227-243, 292-301. Refers to underground placement as "the technique of the Andes." Describes Canon del Pato development, then under construction, and also projects at Rio Mantaro, Mayucallán, and Limeña.

1951

L'AMÉNAGEMENT DE LA CHUTE DE TREIGNAC, SUR LA VEZÈRE. Le Génie Civil v 128 p 11-12.

UNDERGROUND (Letter to the Editor). A. Streiff. Power Engineering v 55 August p 63. Discussion of Authors' Item 51.

1952

UNDERGROUND POWER STATIONS IN SCOTLAND. The Engineer v 194 p 329. A restudy of the Glen Moriston scheme led to the substitution of underground for conventional power stations.

1953

L'AMÉNAGEMENT HYDROÉLECTRIQUE DE L'AAR DANS LE HASLI SUPÉRIEUR; LES NOUVELLES CENTRALES SOUTERRAINES DE HANDECK II ET DU GRIMSEL. Le Génie Civil v 130 p 372-373. Description, map, and profile. Two references to other papers.

LES CENTRALES HYDROELECTRIQUES SOUTERRAINES. Hausser. La Houille Blanche v 8 p 211-212. Abst. of Item 83 and of a paper by Lewin in the same volume of Le Monde Souterraine.

1954

UNDERGROUND POWER PLANTS. Siegfried W. Markl. Water Power v 6 p 442. Abst. from Die Wasserwirtschaft v 43 p 94. Studies a proposed pumped storage installation in Germany with the power plant in a hill between two reservoirs.

L'USINE HYDROÉLECTRIQUE SOUTERRAINE DE BRAZ DES CHEMINS DE FER FÉDÉRAUX AUTRICHIENS. Le Génie Civil v 131 p 458. Abst. from l'Oesterreichische Bauzeitschrift of May 1954.

1955

L'AMÉNAGEMENT DE LA CHUTE DE MONTPEZAT. Le Génie Civil v 132 p 101-104. Description of the project which includes a tapped lake as a reservoir and the diversion of water from an Atlantic watershed to the Rhône River.

1956

EVOLUTION DES SCHEMAS D'UTILIZATION DES COURS D'EAU EN FONCTION DES PROGRÈS RÉALISÉS DANS LES CONSTRUCTIONS SOUTERRAINES. (abst.) Carlo Semenza. Revue Générale de l'Électricité v 65 p 266-267. Abstract of Report IIB, Congress of the International Union of Producers and Distributors of Electric Energy, London, Sept. 1955. Discussion of general principles with Italian and French examples.

1957

L'AMÉNAGEMENT HYDROÉLECTRIQUE DE LA GRANDE DIXENCE (VALAIS, SUISSE). Le Génie Civil v 134 p 433-446. Fionnay and Nendaz plants.

MAITHON UNDERGROUND HYDRO-ELECTRIC POWER STATION. K. M. Chinnappa. Indian Journal of Power and River Valley Development. Special number "Maithon Dam" p 29-38, 50 + plate.

GENERAL FEATURES OF UNDERGROUND POWERHOUSE AT MAITHON. Previous reference, p 39-42 + plate.

WATER POWER PROJECTS—AUSTRALIA. Water and Water Engineering v 61 p 407. Cites plans for 400,000-ph station supplied from Great Lake to underground station about ten miles from Cressy, Tasmania. To be completed about 1963.

The following misprints and omissions were noticed.

p 1350-5, Item 5. "UNCAS" should be "Un cas," "particular" should be "particuliere" and "souteraine" should be "souterraine."

p 1350-7, Item 27. "grande" should be "grand."

p 1350-10, Item 64. "ame'nagement" should be "aménagement."

p 1350-12, Item 83. Complete to read "p 730-735."

p 1350-18, Item 138. "performation" should be "perforation."

p 1350-19, Item 153. "hydro-electrid" should be "hydro-electric."
Item 154. "Qued" should be "Oued."

p 1350-20, Item 163 "souterraines" should be "souterraine" and "travau" should be "travaux."

p 1350-22, Item 176
p 1350-23, Item 190 } "Qued" should be "Oued."

p 1350-30, Line 1 "France" should be "Italy."

p 1350-36, Item 213. Add "Part I, p 593-632."

The following paper was cited in A but not repeated in D.

UNDERGROUND HYDROELECTRIC POWER PLANTS HAVE ECONOMIC AND OPERATIONAL ADVANTAGES. Civil Engineering v 20 p 100-101.

STAVROS N. NICOLAOU,[1] A.M. ASCE.—The authors are to be congratulated for their effort in presenting an up-to-date chronological bibliography including brief resumés of the world's underground hydroelectric plants. The list of the underground hydroelectric plants with general data and references as presented under Section C is most useful to engineers engaged in the design of such developments.

The authors state in their brief discussion of Section B that the basic factor in the selection of underground developments is economic. The writer concurs with this conclusion and would amplify it that the economic factors are not only labor and material costs but also the annual charges for maintenance and operation.

Comparisons of the annual charges for maintenance and operation between above-ground and underground developments indicate savings of between 0.4% to 0.8% in favor of the underground developments.

Recent improvements in (a) preliminary geologic investigations, (b) design analysis, (c) construction equipment, (d) construction techniques in tunneling and steel lined shafts, have combined to effect appreciable reductions in costs of underground power plants.

1. Project Engr., Tippetts-Abbett-McCarthy-Stratton, New York, N. Y.

These lower costs have accelerated the trend towards such developments in recent years. Underground installations account for half of the hydro developments in Sweden. In Italy it is anticipated that by 1960 the installed capacity of underground power stations will reach 4,500,000 KW.(1) Even in the United States underground developments are becoming economically more attractive and hence more competitive. An example is the Haas Project the first large-capacity development ever to be built in the United States, and which will be in operation by 1958.

The authors have carefully prepared Section C indicating the basic data on plant capacity, number and type of units, static head and power station chamber excavation dimensions. Section C would be of even greater value to the engineer if it were supplemented by data showing the excavation dimensions for the housing of the valves, transformers, and turbines for horizontal installations when separate valve transformer and turbine chambers are provided.

The writer has computed on the basis of Section C the volume of power station excavation per KW of installed capacity, per unit and has plotted this relationship with respect to static head for each power station in Figs. 1 to 3. The total volume of the power station excavation has been converted to a per unit basis by increasing the number of units indicated in Section C by one to allow for the erection bay which occupies about the space of one unit.

Curves approximating the average of the plotted points have been drawn for Kaplan, Francis, and Pelton turbines respectively. These graphical relationships indicate that the power station excavation is appreciably affected by the type of turbine (which in turn is influenced by the static head) as well as the type of installation.

Specific conclusions that may be deduced from the graphs are as follows:

1. For Pelton turbines, horizontal installations require from 15% to 40% less volume for power station excavation than the vertical installations.

2. For Francis turbines, horizontal installations require slightly less volume for power station excavation than the vertical installations.

3. For heads between 1,000 - 1,500 feet, Francis turbine installations require from 30% to 80% less volume for power station excavation than the Pelton turbine installations.

4. The range between 1,000 - 1,500 feet has usually been considered as a province of the Pelton turbine. However, with modern turbine design and with the savings in underground excavation at high heads there has been a trend toward the adoption of Francis units for heads up to 1,700 feet.

To derive more accurate curves for underground excavation, additional data such as the dimensions of the valve chamber and the span and rise of the roof arches are required.

It would be most helpful if the authors could indicate the machine hall height from the generator floor to the crown of the roof arch for all underground power plants listed in Section C. On the basis of machine hall heights furnished for the Swedish and Portuguese power plants, the ratio of the volume of the machine hall excavation to the total volume of power station excavation has been plotted against static head. The results of this plotting on Fig. 4 indicate that this ratio varies between 0.43 to 0.80 for heads between 36 to 1,100 feet, with an average value of 0.55 to 0.60.

TABLE I

Roof Arch Data	Country and Plant			
	Brazil Cubatao	Canada Kitimat	Cuba Hanaban.	Philippines Ambuklao
Clear Span-Ft.	73.00	86.00	41.33	44.00
Rise-Ft. (to Intrados)	19.60	32.00	13.40	15.20
Equation Intrados (Metric System)	$y=0.0485x^2$	$y=0.058x^2$	$y=0.103x^2$	$y=0.1034x^2$

The following data supplement SECTIONS C and D of the authors.

SECTION C

Country and Plant	Turb. No. & Type	Plant Capac. MW	Stat. Head Ft.	Powerhouse Chamber L x W x H	Valve Chamber L x W x H
CUBA					
Hanaba-nilla	3 Fv	45	870	120x40x85(50)	90x29.5x37
EL SALVADOR					
Guayabo	3 Fh	45	184	169x37x62(49)	-
FRANCE					
St.Pierre Cognet	2 Fv	50	306	138x92x85(42)	-
PHILIPPINES					
Ambuklao	3 Fh	75	572	230x42x66(50)	118x34x52
Binga	4 Fv	100	510		-

Table I and Fig. 5 furnish data on roof arches of some underground power stations.

SECTION D

1954

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218. CHUTE-DES-PASSES CONSTRUCTION DRAMA, Arthur J. Fox, Jr. Engineering News Record Vol. 159, No. 20, Nov. 14, 1957, p. 52-55. Chute-de-Passes project in eastern Quebec. Consists of a 6.0 mi. Power Tunnel from the existing Passes Dangereuse Reservoir, an underground Power Station housing five units 200,000 hp. each, and a 1.7 mi. Tailrace Tunnel.

219. ST. PIERRE COGNET DEVELOPMENT. Water Power Vol. 9, No. 12, Dec. 1957, p. 452-462. General description of the St. Pierre-Cognet Development. Thin Arch Dam - Intake Tower and Supply Tunnel - Semi underground Power Station - Electrical Equipment.

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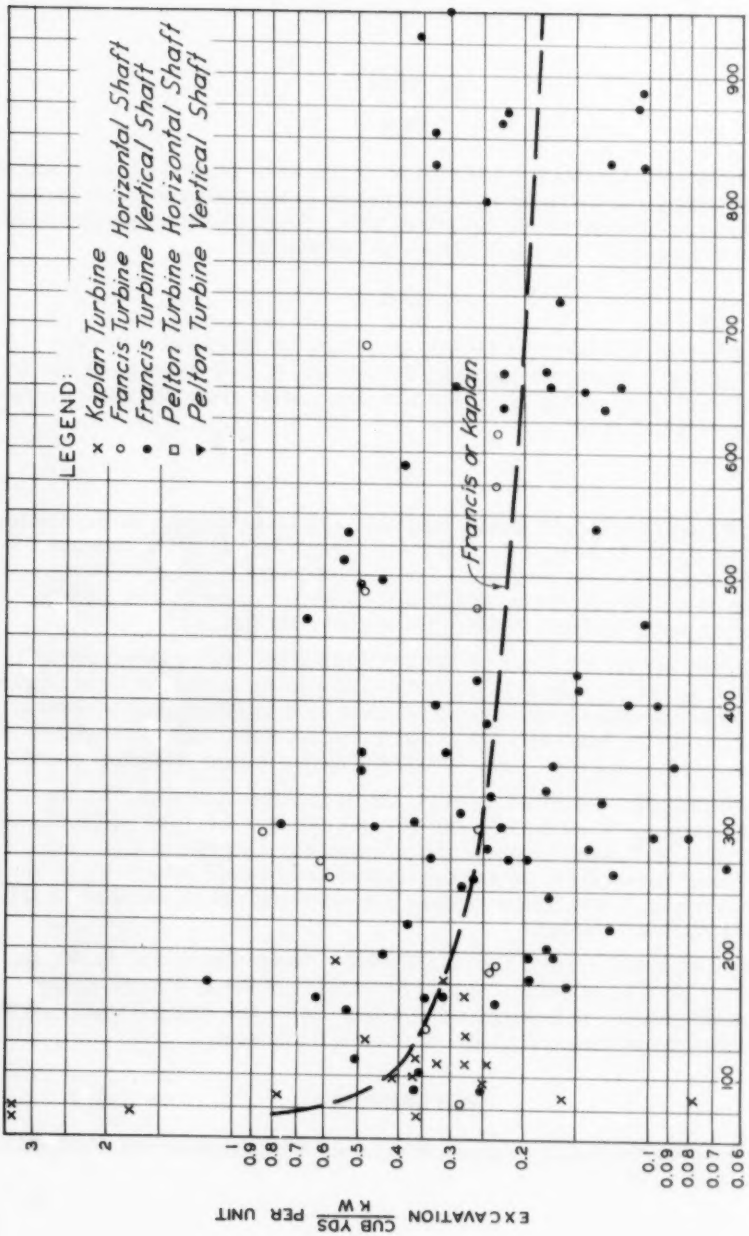

FIG. I APROXIMATE VOLUME OF POWER STATION EXCAVATION FOR HEADS 50-950 FT.

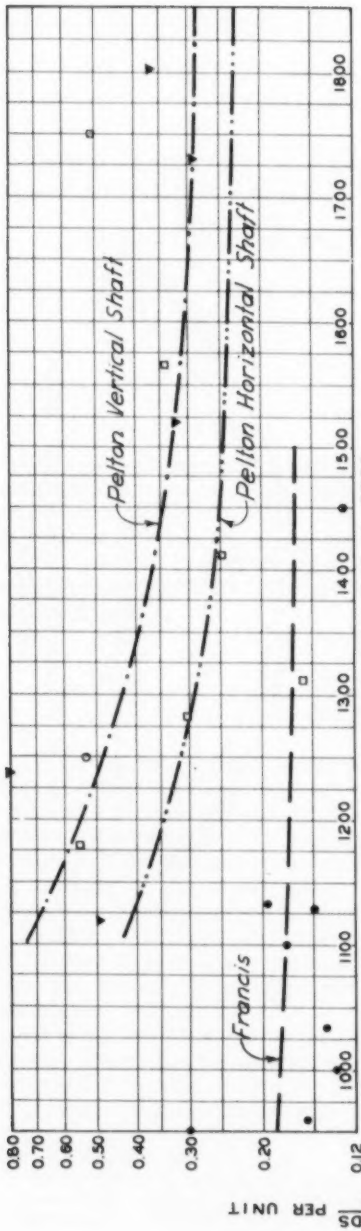

FIG. 2 APROXIMATE VOLUME OF POWER STATION EXCAVATION FOR HEADS 950-1850 FT.

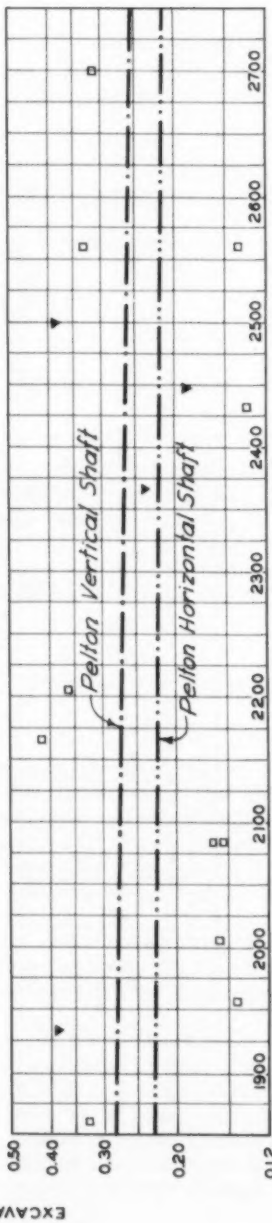

FIG. 3 APROXIMATE VOLUME OF POWER STATION EXCAVATION FOR HEADS 1850-2750 FT.

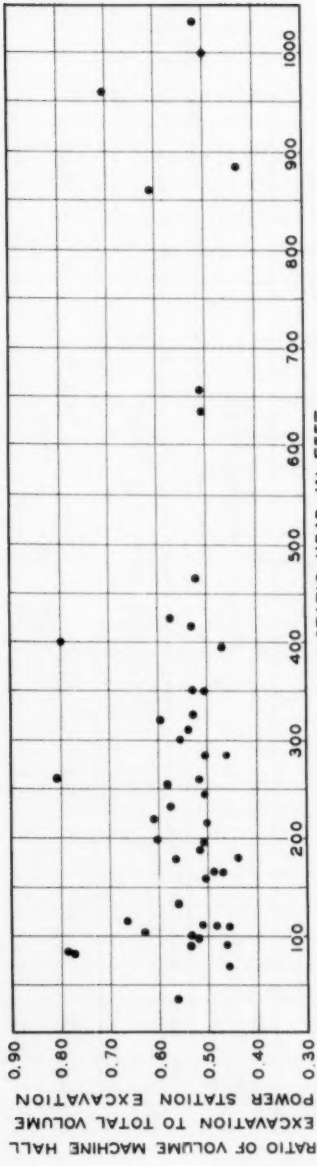

FIG. 4 RELATIONSHIP BETWEEN RATIO OF VOLUME OF MACHINE HALL TO TOTAL VOLUME OF POWER STATION EXCAVATION FOR VARIOUS HEADS

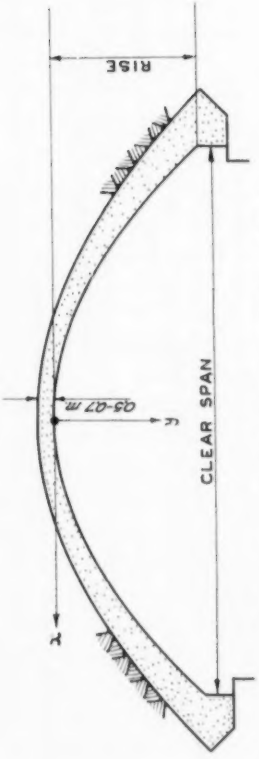

FIG. 5 POWER STATION – ROOF ARCH
See Table I

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TRAJECTORY BUCKET-TYPE ENERGY DISSIPATORS

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(Proc. Paper 1553)

ABSTRACT

Factors to be considered in the design of trajectory bucket-type energy dissipators are enumerated in this paper along with data compiled from model and prototype tests. The wide variance in existing designs precludes the establishment of a specific design procedure until tests for all conditions have been completed. For all contemplated structures recourse to laboratory model studies should be made in order to refine or verify the merits of each individual design.

INTRODUCTION

Trajectory bucket-type energy dissipators are utilized when the tailwater depth at a dam site is considerably less than that required for the formation of a hydraulic jump. When insufficient tailwater occurs, a deep and usually expensive hydraulic jump-type stilling basin will be required. Performance of a trajectory bucket does not depend upon the tailwater depth or any tailwater at all. Consequently, this device may provide a considerable saving over a hydraulic jump-type stilling basin.

Buckets of the trajectory type can be generally classified as being high or low, depending upon the location of the bucket with respect to the riverbed. Figs. 1(a) and (b) depict typical low and high trajectory bucket-type energy dissipators, respectively.

Ski-jump spillways can be included in the high bucket category since flows are discharged into the atmosphere, high above the riverbed. This device has performed very well in France, Portugal, and other countries. Design details of ski-jump spillways have been developed primarily as a product of the

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French designer Andre Coyne.

A typical ski-jump spillway is depicted in Fig. 1(c). This device has been used in conjunction with gravity arch dams, where one or two individual channels of relatively narrow widths direct the high-velocity flow high into the air and remove the erosive impact of the water away from the toe of the dam. Water flowing down the spillway is given a rotary motion such that during its flight, through the air, it is spread out fanwise in height, but not in width. When twin ski-jump spillways are employed they are usually positioned so that the jets will converge thereby effecting energy dissipation. For some structures that have been constructed, the spillways were also designed to serve as the roof of the powerhouse. The thick concrete slab and other structural features provides protection for the powerhouse against air attacks.

Tunnel spillway deflectors are generally located at or slightly above the streambed elevation and can be treated in the same manner as low trajectory buckets. A typical tunnel spillway deflector is depicted in Fig. 1(d). When possible, tunnel spillways are detached from the main dam to avoid endangering the main structure should serious scour occur. Twin tunnel spillways are usually positioned in such a manner that the issuing jets will intersect in the center of the stream.

Typical Bucket Action

High-velocity flows at the base of a spillway are deflected by the trajectory bucket high into the atmosphere and far downstream. As the jet falls freely through the air it will be affected by internal turbulence, shearing action of the air surrounding it, and surface tension. Should the jet trajectory be very long, the combination of the internal turbulence and shearing action may cause the jet to be almost completely disintegrated before it strikes the channel bottom.

Impact of the partially disintegrated jet on the channel bottom will cause a large amount of scour in the vicinity of the impact area. To prevent the scour from progressing upstream and possibly endangering the safety of the structure, the riverbed must be able to withstand considerable scouring. At most dam sites, rock conditions in the downstream channel will be required.

Operation of a trajectory bucket is characterized by the formation of a large amount of spray and a very rough water surface in the downstream channel. These characteristics may be undesirable when powerhouses or other structures are in close proximity to the spillway.

Design Considerations

Factors to be considered in the design of trajectory bucket-type energy dissipators include the bucket shape, entrance and exit slopes, elevation of the bucket lip, lip height, trajectory length, training walls, and erosion of the downstream channel bed. Variables to be considered in the design are shown in Fig. 2.

Bucket Shape

Almost all of the trajectory buckets that have been designed and constructed in the United States are circular in shape. No set rules have been established for determining the bucket radius. For most structures, bucket

radii ranging from 30 to 60 feet will be adequate. It is important that the radius be sufficient to prevent excessive splashing, vibration, or a rough water surface.

For some spillways, particularly those with steep downstream slopes, buckets parabolic in shape may provide better flow conditions than circular buckets. To prevent any disruption of the flow, the parabolic curve should be designed for the maximum velocity in the transverse section at the beginning of the trajectory. The parabolic curve should be conservative to assure that the flow will not spring free of the bucket surface.

Ski-jump spillways, which have become popular in France and other countries, consist of surfaces made-up of composite circular and parabolic vertical curves. Laboratory models were used to determine pressures on the spillway surface. Each design was then carefully checked for simulated operating conditions including both controlled and uncontrolled flow. In order to minimize erosion of the channel banks, the ends of the bucket slab can be tilted so that the flow will be directed away from the banks and into the center of the channel.

Tunnel spillway deflectors consist of simple curved surfaces with a lip height and exit slope to direct the flow away from the outlet portal. In most cases deflector radii ranging from 250 to 400 feet will be found satisfactory. Exit angles, ranging from 5 to 20 degrees, will provide adequate deflection. Deflectors utilized at the Bhakra and Hungry Horse Dams have radii of 390 and 377 feet, respectively.

Deflector buckets at the Fontana Dam[12] are composed of cylindrical spoon-shaped surfaces. Satisfactory results predicted by laboratory model studies were later verified by prototype operation. The final shapes of the two buckets were determined by trial of a number of different designs.

Entrance Slope

The slope of the spillway section entering the bucket is not critical. However, the slope selected should not be so steep that erratic uneven flow occurs. Probably the steepest spillway slope that should be used is 4 vertical to 1 horizontal. Model studies can be utilized to determine the steepest slope that can be used satisfactorily.

Exit Angle, ϕ

The exit angle is a principal factor in determining the jet trajectory length. Adequate deflection can be provided with exit angles ranging from 20 to 40 degrees. Although greater lip angles will deflect the jet farther downstream, they will also produce more spray, require more concrete, and therefore be more costly. Flatter exit angles are usually unsatisfactory since they do not deflect the flow far enough away from the dam. Comparisons between the jet trajectories and erosion, indicated by the model studies, will be helpful in selecting the optimum exit angle.

Bucket Lip Elevation and Height

The elevation and height of the bucket lip with respect to the bucket invert also influences the jet trajectory length. Although no set rules have been established for the height of the bucket lip, it is important that the lip height be sufficient so that the flow will be well directed away from the structure. This

will be somewhat affected by the type of bucket. It is suggested that the lip height be made equal to a portion of the bucket radius as given in Eqs. (1) and (2).

For high buckets

$$b = 0.1R \tag{1}$$

For low buckets

$$b = 0.125R \tag{2}$$

Eqs. (1) and (2) were based primarily on existing designs.

For high buckets it is suggested that the lip elevation be located approximately at the maximum tailwater elevation. In the case of low buckets, the lip elevation should be set high enough above the riverbed to prevent bed material from being carried back into the bucket.

Dentated Lip

It may be desirable to dentate the bucket lip in order to distribute the flow over a larger area of the riverbed than distributed by a plain curved bucket lip. Alternate ridges and troughs will be produced in the flow as it is spread longitudinally by the slotted bucket lip. Experiments conducted for the Cleveland Dam[2] ski-jump spillway indicate that the size of the corrugations will increase when the discharge is increased.

Slotted buckets have been called "double jet deflectors" by Uppal and Singh[13] because part of the flow is directed by the splitters and shoots into the air, while the remainder flows through the slots and is deflected into the air as independent jets. Due to the flaring extensions, jets from the adjoining sections interact with each other thereby dissipating a substantial portion of the initial energy. In contrast to a solid bucket lip it was noted that the depth of scour was reduced, on an average, by 50 per cent when a slotted bucket lip was used.

Sub-atmospheric pressures sufficient to cause cavitation may occur on the surfaces just below the sharp edges of the flow splitters. Cavitation can be reduced or virtually eliminated by rounding the edges of the splitters or by chamfering the edges.

Jet Trajectory Length

As the jet falls freely through the air it will be affected by internal turbulence, shearing action of the air surrounding the jet and surface tension. Weber[16] states that the effect of air resistance or friction will be proportional to the surface area of the jet and to the square of the relative velocity of the jet to air. The velocity difference between the fluids fosters the growth of waves until eventually the high-velocity jet is disintegrated into the air. This hypothesis is confirmed by Schweitzer.[11] It is possible for the jet to disintegrate so completely that it will be formed only of a heavy spray. This phenomena can be observed at many waterfalls.

The surrounding air is effective in absorbing only a small amount of the energy of the jet. Investigations by Horeni[7] conclude that the jet contributes to energy dissipation mainly by drawing air through the enlarged section. Through its volumetric change the air uses up a considerable portion of the mechanical energy. Actions in the air are similar to those in water although

the greater density of the water will cause the jet to diffuse much more rapidly with distance than with the case of air. Acharya[1] states that the momentum transfer theory can be applied to turbulent jets spreading in still air.

Recourse has been made to model tests to predict the lengths of the jet trajectory for various exit angles, discharges, and flow conditions. Capillary forces, present in models, tend to preserve a smooth jet surface. Large jets with high velocities are generally insufficient to confine the unsteady motion known as turbulence, which leads to the disruption of the stream.

Eq. (3) can be used to compute the horizontal component of the jet trajectory when the bucket is set at or slightly above the riverbed. Admittedly, the equation neglects the disintegration of the jet and air resistance.

$$x = \frac{v_o^2 \sin 2\phi}{g} \tag{3}$$

Eq. (3) can be transformed into dimensionless form by dividing each side of the equation by H, the vertical distance between the maximum pool and the riverbed.

$$\frac{x}{H} = 2\left(\frac{h}{H}\right)\sin 2\phi \tag{4}$$

The variables of Eq. (4) are depicted in Fig. 2.

Measurements made at the St. Etienne-Cantales and Chastang Dams,[8] for flows less than one-half of the maximum discharge, show that approximately 19 to 20 per cent of the total energy is dissipated by interaction of the jet with air. To compensate for the velocity reduction of the jet during its flight and loss of energy, the author suggests that Eq. (5) be used instead of Eq. (4).

$$\frac{x}{H} = 1.9\left(\frac{h}{H}\right)\sin 2\phi \tag{5}$$

Obviously, the maximum value of $\sin 2\phi$ in Eq. (5) is unity. Consequently, the maximum jet trajectory length will occur for a 45-degree exit angle.

Fig. 3 presents theoretical curves of $\frac{x}{H}$ versus $\frac{h}{H}$ for bucket exit angles ranging from 10 to 45 degrees. Experimental data collected by Panasenkov[9] is plotted in Fig. 3. It will be noted that this data agrees very well with the theoretical curves. Data compiled from model and prototype tests for the Pine Flat Dam are also plotted. Additional experimental data collected by De Vito[6] closely approximates the relationship of Eq. (5). Trajectory lengths observed in model tests for exit angles, ranging from 25°45' to 45°45', were slightly greater than yielded by Eq. (5).

The designer may also wish to determine the vertical component or height of the jet trajectory. If air resistance and disruption of the jet are neglected, Eq. (6) can be used to compute the jet trajectory height.

$$y = \frac{v_o^2 \sin^2 \phi}{2g} \tag{6}$$

since $\frac{v_o^2}{2g} = h$, Eq. (6) becomes

$$\frac{y}{h} = \sin^2 \phi \tag{7}$$

Both Eqs. (5) and (7) do not account for changes in the bucket radii, that affects both the height and length of the jet trajectory. Experiments conducted for the Hartwell Dam[15] indicate that a larger bucket radius caused the flow to be deflected slightly higher and farther downstream than did a small bucket radius.

Erosion

Probably the most important concern regarding trajectory bucket-type energy dissipators will be over the possible dangerous erosion of the foundation rock by spillway flows. Factors influencing erosion include the velocity, thickness of jet, angle of inclination of jet entering the pool, depth of tailwater, and characteristics of the material being scoured. The last factor, by far, is the dominant influence. Erosion results from differential pressures which cause particles to be lifted into the stream and then carried away. The importance of obtaining a relatively impervious surface over the impact area is readily apparent.

Studies by Schoklitsch[10] conclude that the discharge per unit of spillway crest length is, by far, the dominant factor in the scour of an easily erodible bed material. By doubling the unit discharge, while keeping the height of fall constant, the depth of scour will be increased by about 50 per cent. In order to develop the same increase in scour, with the discharge remaining constant, the height of fall would have to be increased 6 to 7 times.

Laboratory models can be used to determine the scour for a particular design, at least qualitatively if not quantatively. Refinements in the model are usually made on a trial basis until minimum scour is reached.

For some projects, where the spillway is detached from the main dam, the problem of scour may be of lesser importance. Precautions should however be taken to assure that the erosion will not progress upstream to the extent that the safety of the structure may be endangered. Even if erosion occurs in the downstream channel it would be doubtful that the dam would be in jeopardy until the erosion had been neglected for many years.

Training Walls

Side walls will be necessary to confine the flow while the spillway is in use. It may be desirable to converge the sidewalls, in plan, from the entrance to the exit, to effect good flow conditions and construction economy. Laboratory models can be utilized to predict flow characteristics produced as a result of the convergence.

In testing a particular design, it is important to cover the entire range of contemplated operations including the intermediate steps as well as the maximum and minimum conditions. Elder[19] states that it is possible for the design to be completely unsatisfactory at a reasonably small discharge and yet be entirely satisfactory at the maximum probable discharge. This was true at TVA's Boone Dam where, with a trajectory bucket-type energy dissipator, the worst condition occurred at a discharge of 60,000 cubic feet per second, approximately 40 per cent of the design discharge.

Prototype Characteristics

Prototype data are usually difficult to obtain and few spillway buckets have

experienced large flow releases. This is, of course, brought about by the fact that the flood control storage provided will often contain the flood of record and only rare floods will produce major spillway discharges. Consequently, a number of years may elapse before many projects experience large flow releases.

Described herein are several model-prototype conformances and the conclusions which may be drawn from prototype performances. Table 1 lists the characteristics of 9 trajectory bucket-type energy dissipators.

A low trajectory bucket-type energy dissipator is employed for the Conowingo Dam, located on the Susquehanna River in Maryland. The final design included a trajectory bucket with a 12.5-degree exit angle. Records of the Susquehanna Electric Company indicate that the device has operated satisfactorily for over 25 years. During the 1936 flood, the spillway discharged 830,000 cubic feet per second for two to three hours duration. Only a small amount of erosion occurred below the spillway bucket.

Final designs of the Safe Harbor Dam bucket were based upon extensive model tests. The adopted design consists of a 20-foot apron, with a 30-foot radius bucket, and a 27-degree exit angle. Model studies predicted that erosion would start 20 feet downstream from the apron and continue 140 feet from the apron end, with a maximum depth of scour of 47 feet.[5] A discharge of 795,000 cubic feet per second, 72 per cent of the design discharge, was experienced without any noticeable erosion.

Pine Flat Dam,[14] located on the Kings River in northern California, employs a spillway trajectory bucket. To date the maximum discharge experienced by the bucket has been 8880 cubic feet per second (52 days over 7000 cubic feet per second). With a discharge of 8880 cubic feet per second, the jet was deflected approximately 300 feet downstream. This agrees fairly well with the theoretical trajectory length curves of Fig. 3. Fig. 4 is a photograph of the Pine Flat Dam trajectory bucket in operation. Considerable erosion of the riverbed occurred near the impact area of the jets.

A list of ski-jump spillways and their characteristics is shown in Table 2. This list does not include the ski-jump spillways for which no data is available. Among those spillways proposed are the Grandval, Castelnou-Lassouts, Golinhac, Le Yate, La Meurthe, and Monteynard Dams in France; Ponte Pia in Italy, Derbendi Khan in Iran, Oberon in Australia, and San Pedro Piedra Gordo in Mexico.

The first ski-jump spillway was probably constructed in France at the Mareges Dam.[4] Flows from the Mareges spillway are thrown high into the air and a hundred or more feet downstream. After 15 years of operation, with flood flows ranging from 500 to 1500 cubic meters per second, water has dug a large pit in alluvium and granite rock. A pit, 20 meters deep, 60 meters from the spillway lip, was formed with a moveable bottom that takes a new equilibrium shape after each flood. No tendency toward regressive erosion has been found. Downstream from the pit, a bar 6 meters high was formed as a result of the expelled bed material.

Twin ski-jump spillways were placed symmetrically about the L'Aigle Dam[18] centerline, and are designed to convey a maximum discharge of 2000 cubic meters per second. A photograph of the twin spillways is shown in Fig. 5. Due to the lack of space in the narrow gorge, the spillways were shaped to direct the jets of water into the center of the stream and away from the gorge walls. This was accomplished by tilting the discharge ends of the slab until the inside edges were 10 feet higher than the outside edges, at the same

time giving the spillway something of a spoon shape. Flow down the spillway rotates or twists thereby increasing the height while decreasing the width of the discharge jet. With a discharge of 1000 cubic feet per second, the flow was directed 165 feet downstream. For economy reasons, the spillway was also designed to serve as the powerhouse roof. The front of the powerhouse is always accessible during spillway discharges, which lower the tailwater level by a water blast affect. Huge waves with violent sprays occur in the impact area, while the water is quite smooth at the foot of the powerhouse. Overflows of 28,200 cubic feet per second for 20 hours duration were followed by a 20-foot lowering of the riverbed into alluvial strata overlying the hard gneiss rock.

The Chastang Dam[17] is also equipped with two ski-jump spillways capable of discharging a total of 140,000 cubic feet per second. The spillways also serve as the powerhouse roof and are positioned in such a manner that the jets will intersect in the center of the stream.

A single ski-jump spillway was employed for the Cleveland Dam,[2] constructed on the Capilano River near Vancouver, British Columbia. The lip of the spillway was slotted to form splitters which spread the flow and deflected it high into the air for all discharges. With a maximum design of 43,000 cubic feet per second, erosion will take place far enough downstream to preclude any damage to the structure.

Tunnel spillway deflectors are used at many projects including the Bhakra, Lucky Peak, Fontana, and Hungry Horse Dams. The operation of tunnel spillways has been confined mostly to emergency or infrequent use.

Flows from the tunnel spillway of the Lucky Peak Dam,[3] located on the South Fork of the Boise River near Boise, Idaho are controlled by six vertical slide valves in a six-branch manifold. Releases from the tunnel are directed by six individual deflector buckets into an unlined stilling basin excavated in the bed rock and overburden in the river channel. Use of the deflector buckets was dictated by a desire to avoid an expensive lined stilling basin. During 1955, releases up to 5000 cubic feet per second were made through five gate at openings of two feet for 13 days. Although observed conditions indicated satisfactory dispersal and stilling action of the jets, it was noted that erosion of the concrete invert and walls downstream from the gates resulted.

Two unique tunnel spillway buckets were designed for use at the Fontana Dam,[12] located on the Little Tennessee River in North Carolina. Tailwater depths at the tunnel outlet were found to be insufficient to form a hydraulic jump and a trajectory bucket was preferred over a deep expensive concrete-lined stilling basin. Based upon extensive experiments, two buckets, composed of cylindrical sill surfaces, were adopted. Rocks weighing 6 tons or more were placed, as part of the riprap blanket, where the jets struck the riverbed and banks.

During 1945, conditions were favorable for conducting spillway tests. A discharge of 10,000 cubic feet per second was released through each tunnel and the performance observed both for the tunnels discharging together and also singularly. With a discharge of 10,000 cubic feet per second, the corresponding exit velocity at the tunnel outlet was 120 feet per second. As the water left the bucket it appeared to break completely into spray and was directed about 150 feet vertically into the air. In general model-prototype operations were similar with the exception that considerable more spray existed in the prototype. Cross-sections of the riverbed were made before and after prototype tests. Approximately 14,400 cubic yards of bed material were

removed from the riverbed and deposited farther downstream. This has, in no way, affected the safety of the structure. Studies also indicated that the prototype erosion was not as great as predicted by the model.

Summary

An attempt has been made to outline in general terms the factors to be considered in the design of trajectory bucket-type energy dissipators. Emphasis has been placed on the functions, types, characteristics, and operations of trajectory buckets. For all contemplated structures recourse should be made to laboratory model studies in order to determine the merits of each individual design.

The performance of existing structures has been discussed in this paper because of their interest and the possibility that the results may be of assistance in improving the design of future structures. It is important that we continue to observe the performance of prototype structures over a wide range of operating conditions. This will aid the designer as well as the laboratory engineer to extend the range of usefulness of hydraulic models.

ACKNOWLEDGEMENTS

The material presented herein is based both upon experimental and prototype data collected by numerous federal agencies, individuals and hydraulic laboratories. An effort has been made to acknowledge by reference the works of others. In particular the author is indebted to the Laboratoire National D'Hydraulique, Electricite De France, Paris, France for supplying all of their reports on the subject and to the Sacremento District of the Corps of Engineers, U. S. Army for supplying prototype data.

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Table 1
Trajectory Bucket-Type Energy Dissipators

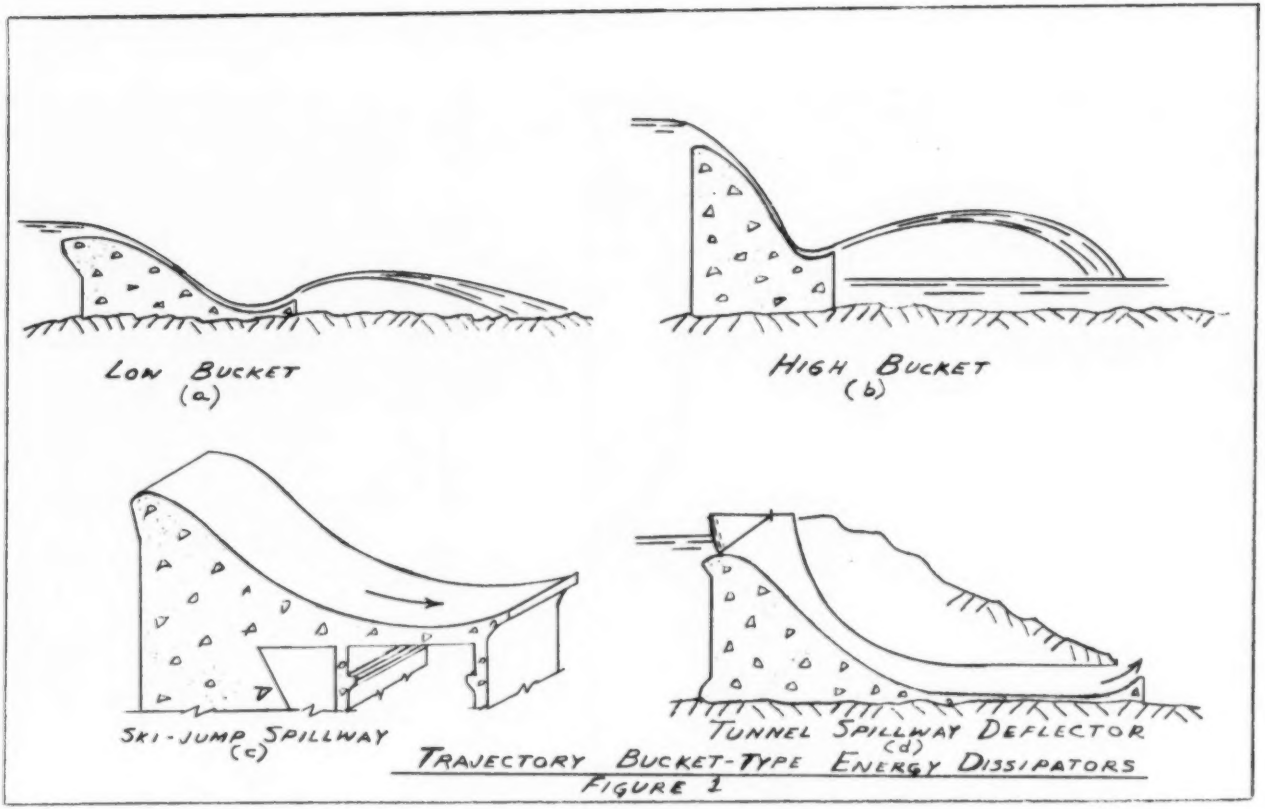
Structure (1)	Location (2)	Bucket Type (3)	Q (c.f.s.) (4)	q (c.f.s./ft.) (5)	H (feet) (6)	$\frac{h}{H}$ (7)	Exit Slope (8)	R (feet) (9)
Anchor	Wyoming	high	13,500	193	102	0.44	14°	parabolic
Arkport	New York	low	29,100	341	--	--	45°	10.2
Chilhowee	Tennessee	low	230,000	865	71	1.0	20°	32
Conowingo	Pennsylvania	low	880,000	391	92	0.76	12.5°	40
Dnieprostroy	U.S.S.R.	high	835,000	334	--	--	--	37.9
Hartwell	Georgia	high	565,000	975	161	0.57	30°	30
Hirakud	India	low	1,187,000	697	130	0.91	40°	50
Pine Flat	California	low	395,000	1,350	372	0.89	20°	50
Safe Harbor	Pennsylvania	low	970,000	365	59	0.76	27°	42

H - vertical distance between maximum pool elevation and riverbed.
h - vertical distance between maximum pool elevation and bucket invert.
R - bucket radius.
q - discharge per foot of bucket width.

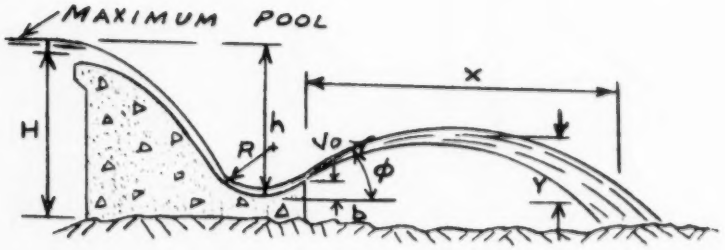
Table 2
Ski-jump Spillway Characteristics

Structure (1)	Location (2)	Number of Spillways (3)	H (feet) (4)	h/H (5)	Q (c.f.s.) (6)	q (c.f.s./ft.) (7)
Bort	France	one	377	0.64	42,200	977
Castelo do Bode	Portugal	two	370	0.82 2/	70,600	2,690
Chastang	France	two	233	0.59	70,400	1,580
Cleveland	British Columbia	one	300	0.62	43,000	540
Genissiat	France		210	0.75	95,000	1,935
Kamishiiba	Japan	two	361	0.30 1/	38,000	600
L'Aigle	France	two	295	0.60	70,400	1,342
Mareges	France	one	253	0.48	24,600	290
Picote	Portugal	one	298	0.66	--	--
St. Etienne-Cantales	France	two	230	0.53	17,600	753

1/ - average.
2/ - right spillway only.
H - vertical distance between maximum pool elevation and riverbed.
h - vertical distance between maximum pool elevation and spillway exit.
Q - discharge for single spillway.
q - discharge per foot of spillway width at exit.



TRAJECTORY BUCKET-TYPE ENERGY DISSIPATORS
FIGURE 1



TRAJECTORY BUCKET VARIABLES

FIGURE 2

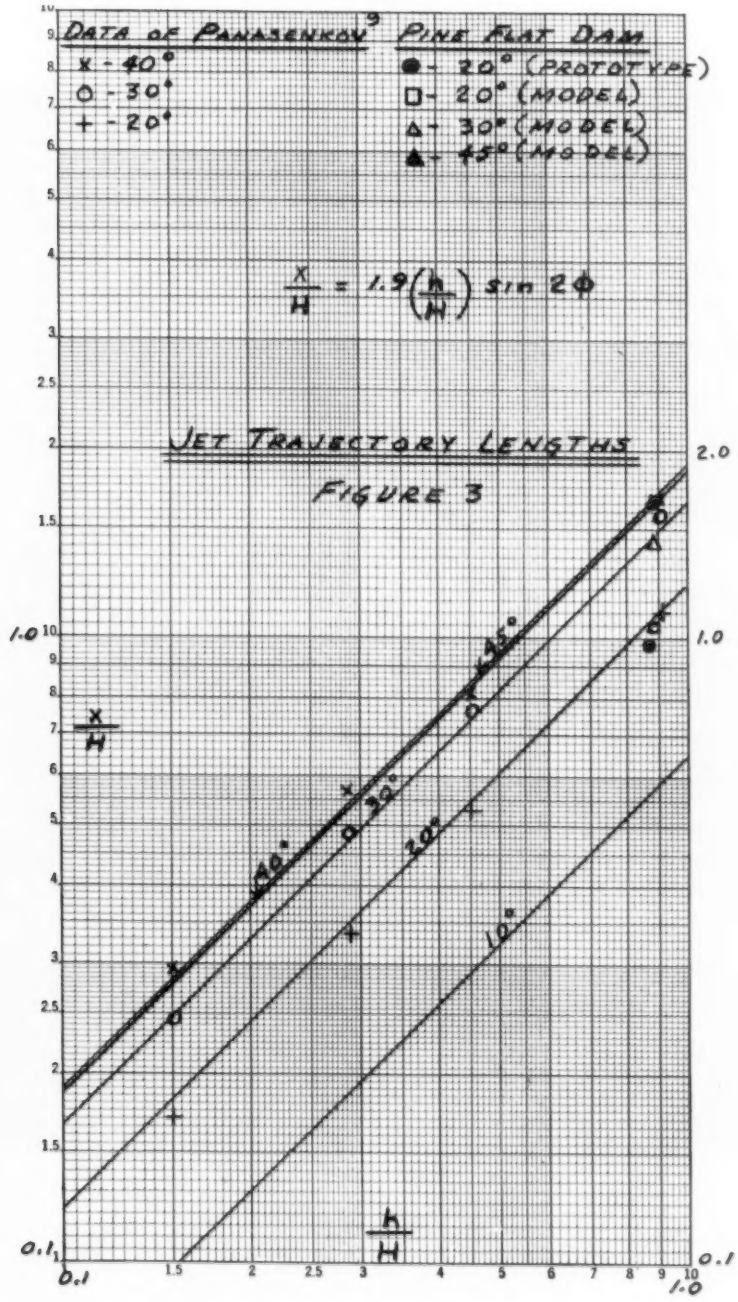

DATA OF PANASENKOV[9]
x - 40°
o - 30°
+ - 20°
PINE FLAT DAM
● - 20° (PROTOTYPE)
□ - 20° (MODEL)
△ - 30° (MODEL)
▲ - 45° (MODEL)
$\frac{X}{H} = 1.9\left(\frac{h}{H}\right)\sin 2\phi$
JET TRAJECTORY LENGTHS
FIGURE 3
$\frac{X}{H}$
$\frac{h}{H}$
45°
30°
20°
10°
0.1
1.0
2.0

Fig. 4. Performance of trajectory bucket-type energy dissipator at Pine Flat Dam, California. (Courtesy of Sacramento District, Corps of Engineers, U. S. Army.)

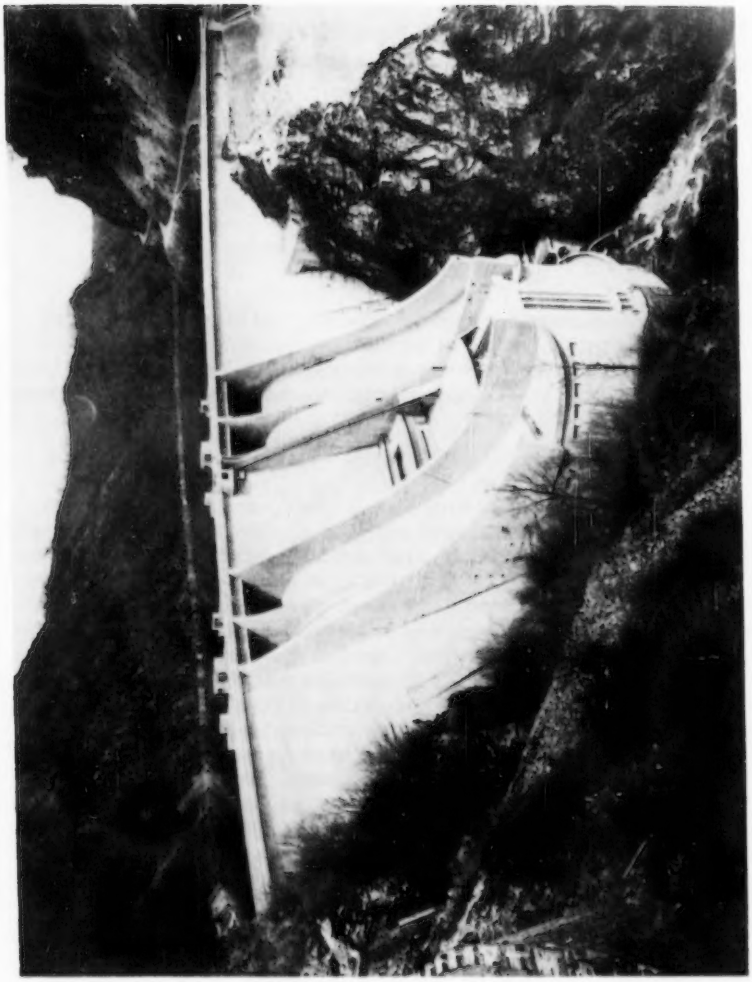


Fig. 5. View showing twin ski-jump spillways of L'Aigle Dam, France. (Courtesy of Constructions Mecaniques et Hydrauliques, Grenoble, France.)

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UNDERGROUND POWER HOUSES IN ITALY AND OTHER COUNTRIES

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(Proc. Paper 1554)

SUMMARY

This paper presents the characteristics of a certain number of underground power houses mainly constructed in Italy to the writer's design or with his cooperation. The fundamental criteria aiding in the choice of the solutions adopted are given special attention.

Preliminary

The first underground power station built in Europe was that of Porjus (Sweden), in the year 1911; in Italy the first example of the kind was the power house of Coghinas, Sardinia, built in 1926 to the design of Ing. Angelo Omodeo. Successively, and especially during the last 20 years, the construction of underground power houses has been considerably favored in Europe. In Italy the power houses of this type, already built or being built, number 60 and others are at present being studied and designed.

The writer has designed and built 10 of these power houses, (out of the 13 belonging to Edison Group of Milan) during the period from 1939 up to now.

A table with various drawings showing the main features and a photo are attached hereto for each of these power houses.

The attached table n°1 contains for the same plants other data and particularly the period of construction and the capacity of the installed machinery. Besides the above mentioned 10 underground power houses constructed for the Edison Group in Italy, the table includes also the one constructed in Brazil (Juquiá development, plant n°1, State of São Paulo) and another still in the design stage, for the Macchupicchu power plant in Peru, both designed by the Hydroelectric Construction Department of the Edison Company of Milan, under the direction of the writer.

For the two power houses of Santa Massenza and Nembia (Italy) owned by the Sarca-Molveno Society, the writer was appointed technical adviser for the design and construction.

Note: Discussion open until July 1, 1958. A postponement of this closing date can be obtained by writing to the ASCE Manager of Technical Publications. Paper 1554 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. PO 1, February, 1958.

1. Cons. Engr. and Technical Director, Società Edison, Milan, Italy.

In addition, this paper contains also the design drawings prepared by the writer in cooperation with Ing. Angelo Omodeo for two projects in Russia: viz. the Niva plant nº3, in the Scandinavian peninsula (Kandalascia Bay) and the Terter plant, in Northern Caucasus, and the data of the Cividate power station designed and constructed by Ing. Fernando Benedetto for a Company of the Edison Group.

Hydroelectric Schemes with Underground Power House

The underground location adopted in a hydroelectric project for the power house and for all or part of the head and tail structures gives the designer a certain freedom from the limitations imposed by topographical conditions and often permits obtaining wider and more economical solutions. Of course, the geological conditions should be such as to allow the construction of important underground works, without excessive cost.

Among the various schemes adopted for projects with an underground power house and herein mentioned, three are the typical ones to which all the others can be referred. They are: 1) the arrangement which was first and more extensively used in the Scandinavian peninsula and was adopted by the writer for Juquiá nº1, Brazil (see fig. 1); 2) the arrangement adopted for Bitto nº4, Italy (see fig. 2); and finally 3) that of Santa Giustina, Italy (see fig. 3).

For Juquiá nº1 the location of the power house practically below the dam site was imposed by topographical and chiefly by geological conditions (a very thick layer of decomposed rock covers the solid granite encountered at lower depth).

The adoption of the underground solution actually permitted setting the machine hall of the Bitto nº4 plant at a lower level than the elevation of the surrounding alluvial plain and reaching the discharge point into the river Adda by a 1,300 m long tailrace driven through the alluvial overburden.

In the case of Santa Giustina, finally, the ground where the plant structures had to be developed consisted of limestone in the upstream portion and marl in the downstream half. The power tunnel had to be excavated through limestone, which gave better assurance in view of the high hydrostatic pressure for which the tunnel was designed, while the steel penstock, the power house and the tailrace were sited in the marls. The presence of marls suggested a curved wall lining, suitable to resist possible external pressures caused by the swelling of the marls.

It is, therefore, the position of the power house with respect to the intake and tail structures that marks the type of development with underground power house. The power house site can be, as above seen:

a) near the intake
b) near the return point
c) halfway between intake and return points.

a) Power house near the intake

This alternative, which is the typical one adopted in the Scandinavian peninsula, may be imposed by the topography of the site as in the case of Niva nº3 and by the geological conditions as for Rio Juquiá plant, and has the considerable advantage of requiring a very short power tunnel, head race

and penstock. It is necessary, however, to keep in mind, always from the point of view of the hydraulic operation, that head losses between the units and the tailrace outlet, in case of free flow tailrace, are practically constant regardless of flow rate changes and equal to the value corresponding to the maximum flow of the power plant.

This scheme may be adopted only with the indispensable condition that the rock have optimum solidity characteristics and be, above all, impervious: otherwise it is evident that serious difficulties could be met in the power house, which is practically adjacent to a reservoir, and in the construction of the tailrace, which may be found to cross water-bearing zones in which water percolates from the stream flowing above.

Consideration should also be given to the problem of the means of access to the power house as well as that of electrical connections, and to the necessity of hauling a large part of the excavated material to a suitable disposal area.

Finally, it is to be taken into account that by adopting such an arrangement, which generally calls for a considerable length of tailrace, a compensating chamber is generally required at the turbine discharge, in order to permit the starting of the units at full load even with water level corresponding to flood level in the river.

b) Power house near the return point

This alternative belongs to the conventional scheme of power plants.

It is worth pointing out that if the power house is sited in the open, the penstock is also generally in the open; in fact a tunnel penstock coming out in the open just at the connection with the units would be hardly justifiable, especially in case of high head and large flows.

When comparing the two possible alternatives with underground or surface power house, in favor of the former there is chiefly the saving obtainable in the penstock and possibly in the machinery connected thereto, while in favor of the latter there is the unquestionably lower cost of the machine hall and access, especially when the foundation may stand on ground where no serious consolidation works are required.

The power houses designed and constructed by the writer according to the described scheme are chronologically the following: Terter and Niva in the U.S.S.R.; Regoledo, Prata, Cedegolo, Isolato, Morasco, Pantano d'Avio and Boazzo in Italy. Also Santa Massenza power house is included in the scheme in question. A quite particular case is Gerola (Bitto n°2) power house, which was built with a reinforced concrete structure and then covered with earth to protect it from avalanches.

The Nembia power house, which is included together with Santa Massenza within the group of hydroelectric plants designed and constructed for the utilization of the River Sarca and Lake Molveno, represents, by reason of its particular shaft structure, a quite singular case and certainly not to be generalized.

In the above described cases the choice of the alternative has always been determined by economic considerations; in no case were war actions considered.

c) Power house halfway between the intake and the return point

Sometimes it may be advantageous to locate the machine hall in an intermediate position between intake and the return point, as already seen for Santa Giustina, which is the only arrangement of this type among the plants herein described.

Notes on the Individual Parts of the Development

Let us now discuss in further details the principles to be followed in the choice of the arrangement of the plant structures.

a) Power tunnel and penstock

The power tunnel which may be pressure or free flow tunnel, is practically independent of the alternative adopted for the machine hall.

The penstock, on the contrary, strictly depends upon the type of power house adopted.

For the alternative with power house in the open also the penstock follows normally, but not necessarily, an alignment totally or partially in the open; for the alternative with underground power house, on the contrary, the alignment of the penstock has obviously to be always in tunnel, at least in the vicinity of the power house.

Of this type are the penstocks of Bitto n°2 and 4 plants, Sonico-Cedegolo, Liro n°1 and Sabbione plants. An example of the described arrangement is given in fig. 4 showing the longitudinal profile of Sonico-Cedegolo power plant penstock.

More frequently, however, the underground power houses have penstocks entirely in tunnel, especially when it is convenient to depend partly on the rock for resisting the water pressure inside the conduit.

Within the limits of head suitable for the adoption of Francis turbines, an underground power house may offer also an operating advantage, and a consequent cost reduction, when the length of the penstock can be reduced so as to avoid the necessity of turbine relief valves, without affecting the conditions of unit regulation (PD^2, governor closure time).

An economic analysis should then obtain the minimum total cost of the two opposing elements, viz. length of the penstock on one side and length of tailrace and access to power house on the other. Such an analysis will be the basis for determining the value of the penstock slope.

Out of the 17 power plants with underground power house referred to hereabove, 7 have a section of the penstock in vertical shaft; the longest vertical shaft excavated in rock, in the cases herein considered, is 194 m long, but, of course, such a length is not to be intended as a top limit.

Other 9 power plants have the penstock in an inclined tunnel. The adopted slope generally has a value from 70 to 85%.

The penstocks of Cividate and Vinadio power houses are in the open, but these power houses, are quite particular cases.

The placing of steel penstocks in the tunnel may be carried out in two ways: by laying them on supporting saddles and anchoring the pipe at the intersection points of the alignment and at intermediate points of long straight sections, as normally done for penstocks in the open, or by filling the space between the penstock and the rock with concrete, so that this may

transfer to the rock part of the load acting inside the penstock. Of course, in this last case it is necessary to have a particularly solid and sound rock and to provide a suitable drainage system to protect the penstock during the emptying periods, against possible external water pressures.

To this end the solution shown in the Bitto n°4 drawings was adopted by the writer since 1939, consisting of an inspection and drainage duct parallel to the piping.

Attention should also be given, however, to some minor advantages pertinent to the installation of a free penstock in tunnel such as a greater facility and speed in placing and full accessibility of the piping.

When the penstock is not subjected to a very large hydrostatic pressure (in the order of about ten atm.) reinforced concrete pipes, either precompressed or not, may be taken into consideration, although in the cases hereabove referred to and which could be suitable for their use, the alternative with steel piping was found to be always the most favorable, especially in view of the higher safety of operation.

b) Relative position of the penstock, machine ball and tailrace

The position of the penstock should be in a plane parallel to the longitudinal axis of the machine hall. The most advisable arrangement, which has been adopted by the writer in many power plants, is to place the distributing conduit at such an angle in the space as to reach the machines with flatter angles and shorter length of piping.

The tailrace is generally arranged longitudinally and opposite to the penstock outlet; in some cases, on the contrary, the pipeline can be sited above the tailrace itself, chiefly for moderate flows.

Another arrangement frequently adopted by the writer is that providing, in case of breakage in the penstock or in its distributor, the possibility of discharging the water without flooding the power house. To this end a gallery by-passing the power house should be provided, with a calibrated opening allowing for control of the maximum discharge.

Similar arrangements have been adopted for the power plants of Santa Giustina, Juquia n°1, Liro n°1, Macchupicchu and others.

For Boazzo it was possible, on the contrary, to adopt the other above mentioned alternative with penstock above the tailrace.

c) Access to power house

The access may be obtained either through a horizontal gallery or through a more or less inclined gallery or even through a vertical shaft.

The choice is generally imposed by the particular local conditions, such as the physical features of the surrounding area, the various water levels in the river in correspondence to the access and the elevation of the power house floor with respect to the outside area which may be utilized for the switchyard and the take-off of the high-voltage outgoing lines.

Most of the above described power houses are reached through a horizontal or slightly sloping gallery.[1]

1. Only for Vinadio and Cividate power houses, owing to their peculiar arrangement and consequent vicinity to ground surface, the machinery could be easily introduced from above.

In addition to the main access, used also for the passage of machines, an emergency exit is generally provided for personnel and can be used also for ventilation purpose.

d) Transformers and electrical connection with the outside

For underground power houses due consideration should be given to the problem of outdoor or underground transformer installation, in which a determining feature is the distance between the machines and the high voltage line take-off.

The advantage of one alternative with respect to the other should be studied case by case with reference to the capacity and number of the machines, besides the length of the connection between the machines and the transformers. Of course, comparisons should be made between outdoor or underground installation costs, besides the cost of connections and their losses, making also allowance for the less effective operation of a generator with increased overall reactance in the generator-bus bar system and the effect on the static and dynamic stability of the network.

In this connection it should be noted that for safety reasons generally the outdoor or underground location of the transformers does not modify the criterion according to which the disassembly of the transformers has to be carried out in a room separate from the machine hall, especially when the machines are many and important. Consideration should also be given to higher cost of underground disassembly room, if any, and relevant fire protection installations which are more expensive for underground than for surface plants.

As regards the electrical connection between transformers and machines, the arrangement with generator voltage bus-bars is more costly than with oil insulated cables and also resistance losses are greater. Moreover, the capability of the generators, as said above, is reduced in view of their greater reactive voltage drop.

Therefore, although the alternative with high voltage connection is nearly as onerous as that with the transformers in the open and connection at generator voltage, consideration should be given to the saving which can be obtained in the transmission network.

In the power plants of Santa Massenza, Boazzo, Sabbione and Bitto n°4, the transformers were placed underground, while in the power plants of Sonico-Cedegolo and Liro n°1, peculiar circumstances dictated the connecting of generators and transformers by a bus-bar system, although the length of the access gallery to machine hall seems, at first sight, to make this arrangement unsuitable.

In the power house of the Santa Giustina plant, an underground location of transformers would have required covering the difference of level of about 160 m between the power house and the surface substation by means of pressure oil cables, which, at the time the power house was constructed, were still in the experimental stage for such type of installation.

These and other reasons have therefore suggested the adoption of a vertical shaft 160 m deep containing the bus-bars at generator voltage.

The 3 groups of bus-bars were segregated in the shaft by means of vertical partitions, while horizontal slabs divide the shaft into 6.50 m high chambers, in order to avoid possible short circuit arcing extending to the whole shaft.

e) Main services

With respect to general services the underground power houses are not substantially different from those in the open, except for the opportunity of providing air conditioning for the machine hall and the particular attention which must be given to the lighting of the rooms.

Air circulation in the rooms may be obtained either by means of natural ventilation or by an air conditioning plant.

It does not seem possible to give a general criterion for choosing one system rather than another, as the problem has to be solved case by case on the basis of the location of the machine hall and of the tunnels departing from it and which could be utilized as ventilation ducts; fundamental factors are the temperature and the moisture relating to the area where the power house is. It is generally preferable to resort to an air conditioning plant.

In almost all the underground power houses designed by the writer, due also to the high elevation at which they are located, the installation of air conditioning was necessary, which provided in every case conditions sufficiently comfortable for the personnel and favorable to the maintenance of electrical equipment.

The light installation in the underground power houses should be planned so as to diminish, as far as possible, the feeling of discomfort often perceived by those who are obliged to spend a long time and to operate in an ambient with artificial lighting. Therefore it should aim at rendering easily visible the control equipment and parts of machinery requiring inspection and also reproduce in the best way the natural light, giving care to its uniformity.

Also a suitable painting of the walls and more visible parts of machinery may contribute to a comfortable ambient.

f) Construction method

The construction of the underground power houses herein described was carried out by the system of galleries and shafts.

Geological conditions and the size of excavation in general made the adoption of full section excavation inadvisable.

Excavation was started with a drift in the top section followed, without delay, by vault excavation and relevant walling. It was deemed advisable, in fact, to construct concrete arch supporting the ceiling rock in all the power houses herein referred to.

Upon construction of the vault the machine hall excavation was carried on either through further drifts and shafts or by blasting and mucking according to the geological conditions and excavation size.

In some cases the rock walls having inadequate resistance had to be consolidated with concrete.

For this purpose were the supporting pillars of the crane tracks generally used, keying them against the foundation structures of the machinery: this system was followed in the construction of the power house of the Sonico-Cedegolo hydroelectric plant.

In other cases it was necessary to carry out the abutment excavation leaving the central core of the rock to serve as support of timbering required during the excavation.

In the event of particularly unfavorable rock conditions, a complete lining of arched pattern was carried out, as necessary in the Santa Giustina power house.

Also when the rock does not call for any lining, it is generally better to separate it from the machine hall proper by means of a partition wall between the pillars of the crane track, so as to form an accessible space where seepage water, if any, may be drained. In this way an air chamber is also obtained which often proves very useful in arranging the air conditioning.

Finally, it may be useful to insist that the longer side of the power house should not be parallel to possible vertical stratification planes of the rock so as to avoid considerable and even unsymmetrical stresses on the vault. It is equally advisable that the distributing conduit and compensating chamber or the tailrace inlet elbow be not too close to the main hall, especially when they are parallel to it and the rock is not particularly solid.

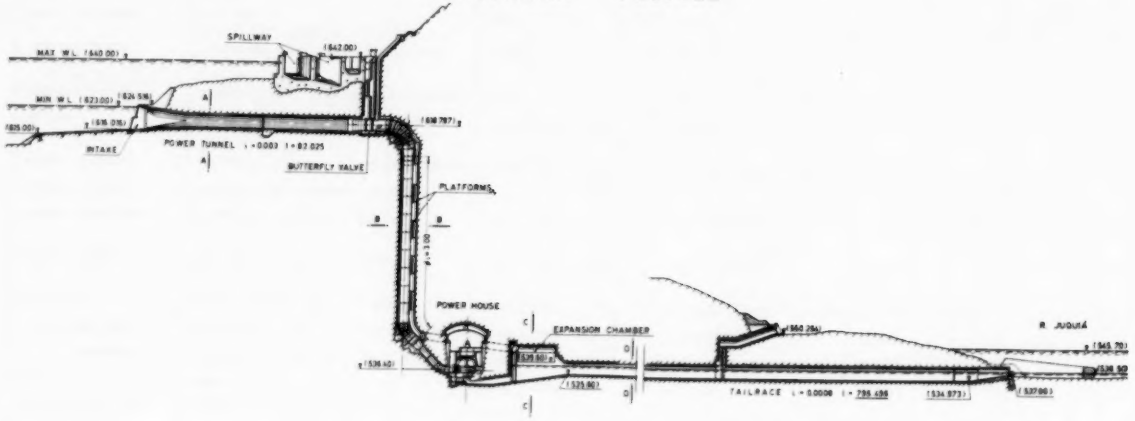
Table N.1

CHARACTERISTICAL DATA OF UNDERGROUND POWER HOUSES

Project	Owner	Design Date	Starting of operation	N.units	Total installed capacity (kW)	Turbines		Excavation volumes (m^3)			Access gallery length (m)
						Type	Axis	Machine Hall	Transf. and other rooms	Access gallery	
Bitto IV plant	Orobia, Milano, Italia	1940	Nov. 1941	2	37.178	Pelton 2 wheels	Horiz.	9.100	2.800	11.600	123
Bitto II plant	Orobia, Milano, Italia	1940	Jan. 1942	1	15.000	Pelton	"	-	-	-	-
Cedegolo-Cividade	Edison, Milano, Italia	1938	Jul. 1942	3	52.500	Francis	Vert.	23.550	-	-	-
Mera I plant	Ediseonvolta, Milano, Italia	1947	Nov. 1949	3	60.450	"	"	10.000	-	1.550	42
Semico-Cedegolo	Industriale Oglio, Milano, Italia	1948	Nov. 1950	3	73.200	"	"	17.850	-	13.900	338
S.Giustina-Taio	Industriale Noce, Milano, Italia	1949	Aug. 1951	3	105.000	"	"	26.550	-	9.700	330
S.Massenza	Sarca Molveno, Milano, Italia	1948	Mar. 1952	9	377.000	Pelton 2 wheels	Horiz.	80.000	20.000	14.000	370
Liro I plant	Ediseonvolta, Milano, Italia	1950	Feb. 1953	2	42.800	Pelton 2 wheels	"	10.000	-	6.700	220
Sabbione-Morasco	Ediseonvolta, Milano, Italia	1950	Mar. 1953	2	44.700	Pelton	"	10.700	6.200	5.000	175
Pantano d'Avio	Ediseonvolta, Milano, Italia	1952	Jul. 1955	2	14.100	"	"	7.500	-	-	-
Stura II plant	Compagnia Imprese Elettriche Liguri Genova, Italia	1954	Mar. 1956	2	61.250	Francis	Vert.	6.250	-	-	-
Nombia	Sarca Molveno, Milano, Italia	1954	under const.	1	13.900	"	"	8.600	-	2.180	72
Bissina-Boazzo	Idroelettrica Alto Chiese, Brescia Italia	1956	" "	2	94.740	Pelton 2 wheels	Horiz.	15.900	11.100	3.000	104
Juquia I plant	Companhia Brasileira de Aluminio, Sao Paulo, Brasile	1954	" "	2	29.740	Francis	Vert.	5.000	-	3.150	176
Niva III plant	U.R.S.S.	1935	-	4	120.000	"	"	22.650	-	5.700	75
Terter	U.R.S.S.	1936	-	3	54.000	"	"	13.300	-	2.780	120
Machupicchu	Corporacion de Reconstruccion y Fomento del Cusco, Peru	1957	-	2	40.000	"	"	4.360	-	3.000	143

HYDROELECTRIC DEVELOPMENT ON THE RIO JUQUIÁ – 1st PLANT (BRAZIL)

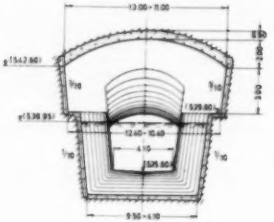
SCHEMATICAL PROFILE



SECTION A-A

SECTION B-B

SECTION C-C



SECTION D-D

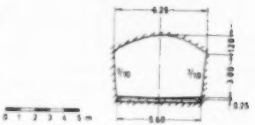
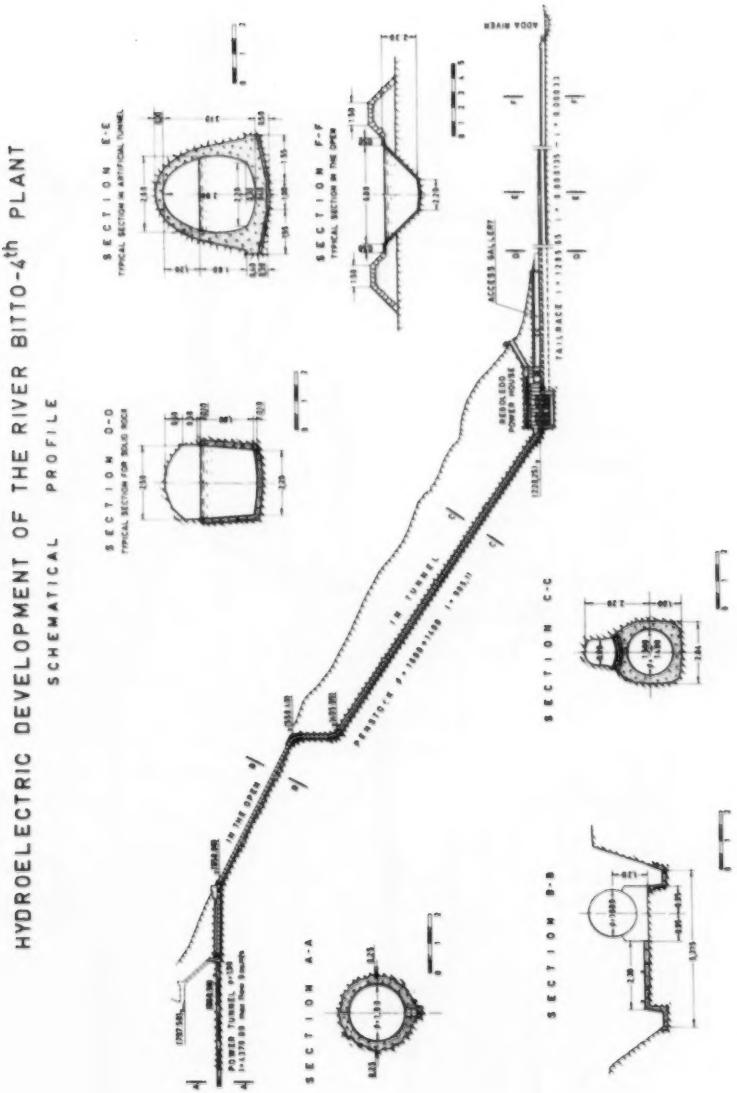


Fig. 1

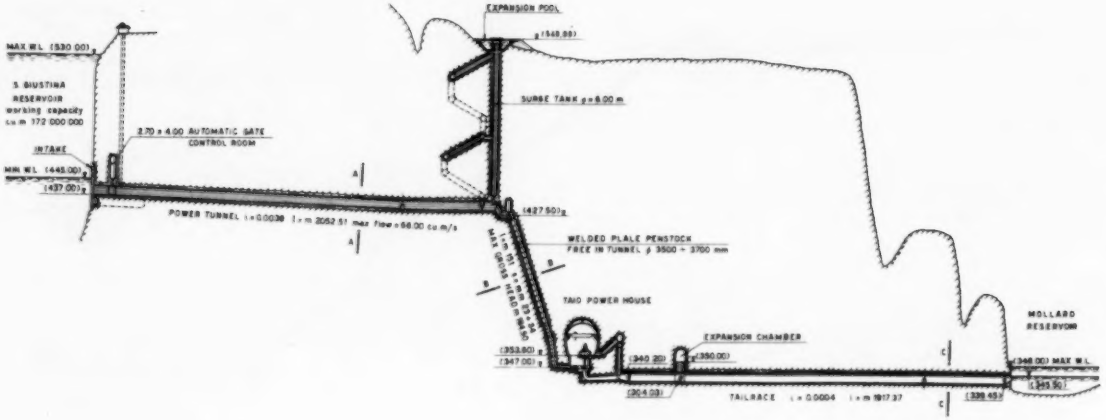

HYDROELECTRIC DEVELOPMENT OF THE RIVER BITTO-4th PLANT
SCHEMATICAL PROFILE
SECTION A-A
SECTION B-B
SECTION C-C
SECTION D-D
SECTION E-E
SECTION F-F
ACCESS GALLERY
TAILRACE

25387

Fig. 2

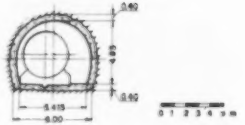
HYDROELECTRIC PLANT ON RIVER NOCE FROM S. GIUSTINA TO MOLLARO

SCHEMATICAL PROFILE


EXPANSION POOL
MAX WL (530.00)
S. GIUSTINA RESERVOIR working capacity cu.m 172 000 000
INTAKE
MIN WL (448.00)
(437.00)
2.70 x 4.00 AUTOMATIC GATE CONTROL ROOM
SURGE TANK ø = 8.00 m
POWER TUNNEL i = 0.0038 l = m 2052.51 max flow = 58.00 cu.m/s
(427.50)
WELDED PLATE PENSTOCK FREE IN TUNNEL ø 3500 ÷ 3700 mm
TAIO POWER HOUSE
EXPANSION CHAMBER
MOLLARO RESERVOIR
(346.00) MAX W.L.
TAILRACE i = 0.0004 l = m 1917.37

SECTION A-A

SECTION B-B



SECTION C-C

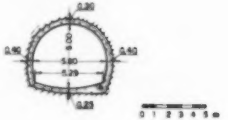


Fig. 3

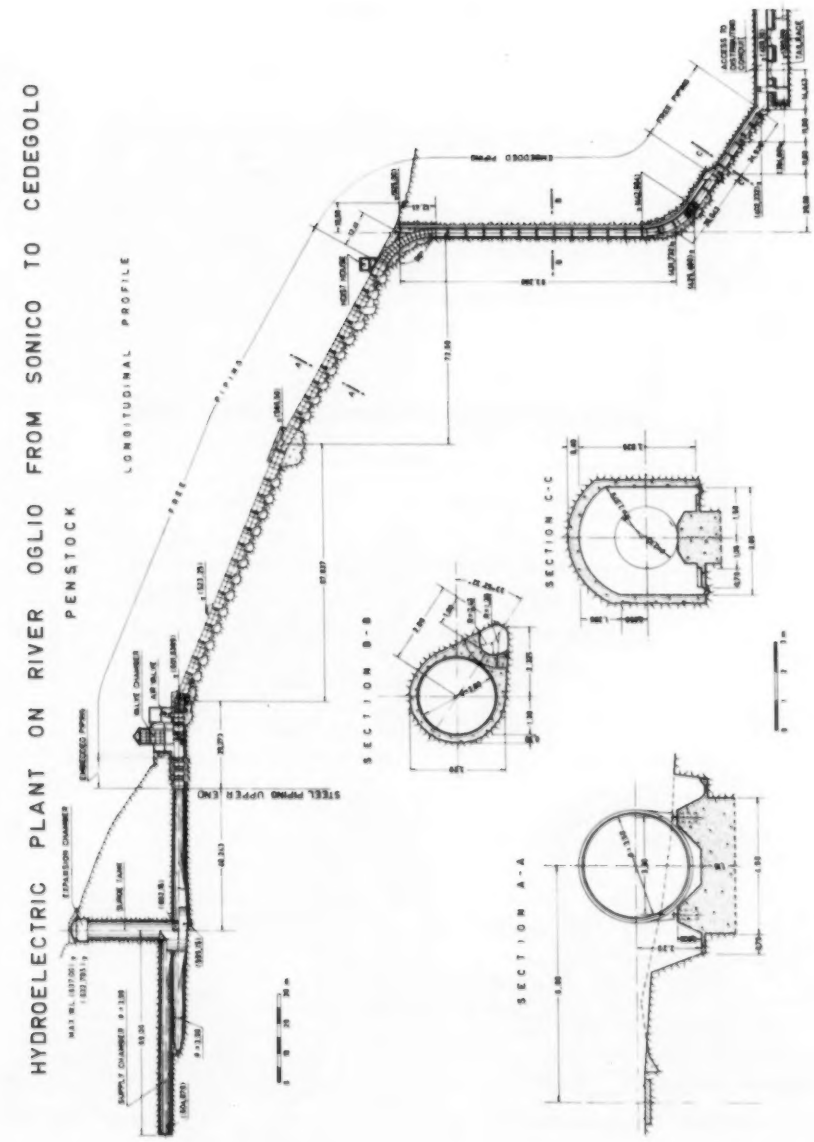
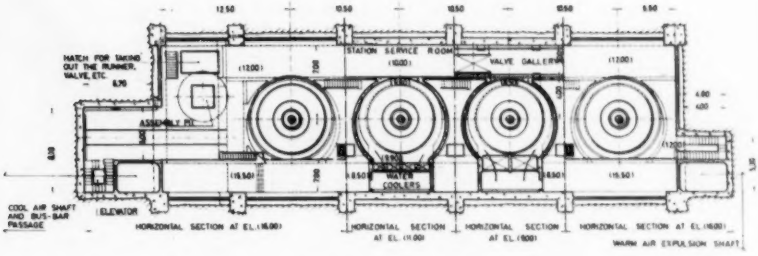

HYDROELECTRIC PLANT ON RIVER OGLIO FROM SONICO TO CEDEGOLO
PENSTOCK
LONGITUDINAL PROFILE
SECTION A-A
SECTION B-B
SECTION C-C

Fig. 4

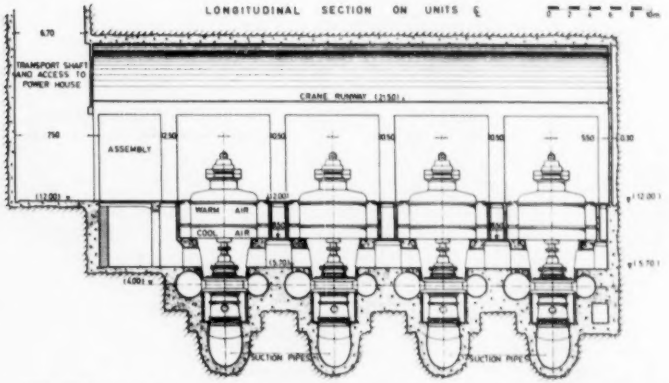
HYDROELECTRIC DEVELOPMENT OF THE RIVER NIVA - 3rd PLANT (U.R.S.S.)

POWER HOUSE

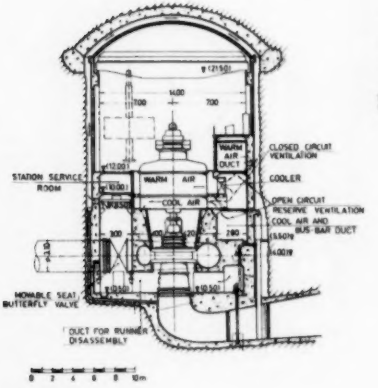
HORIZONTAL SECTION



LONGITUDINAL SECTION ON UNITS ℄

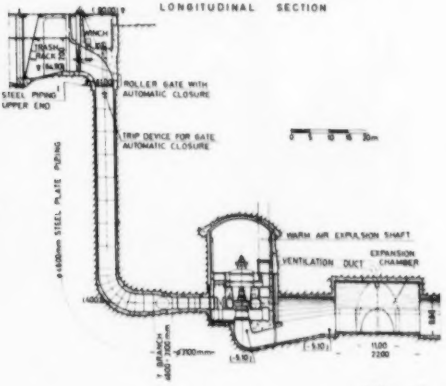


CROSS SECTION ON UNITS ℄



INTAKE - PENSTOCK - POWER HOUSE AND EXPANSION CHAMBER

LONGITUDINAL SECTION



25342

Fig. 5

HYDROELECTRIC PLANT ON THE RIVER TERTER (U.R.S.S.)

POWER HOUSE

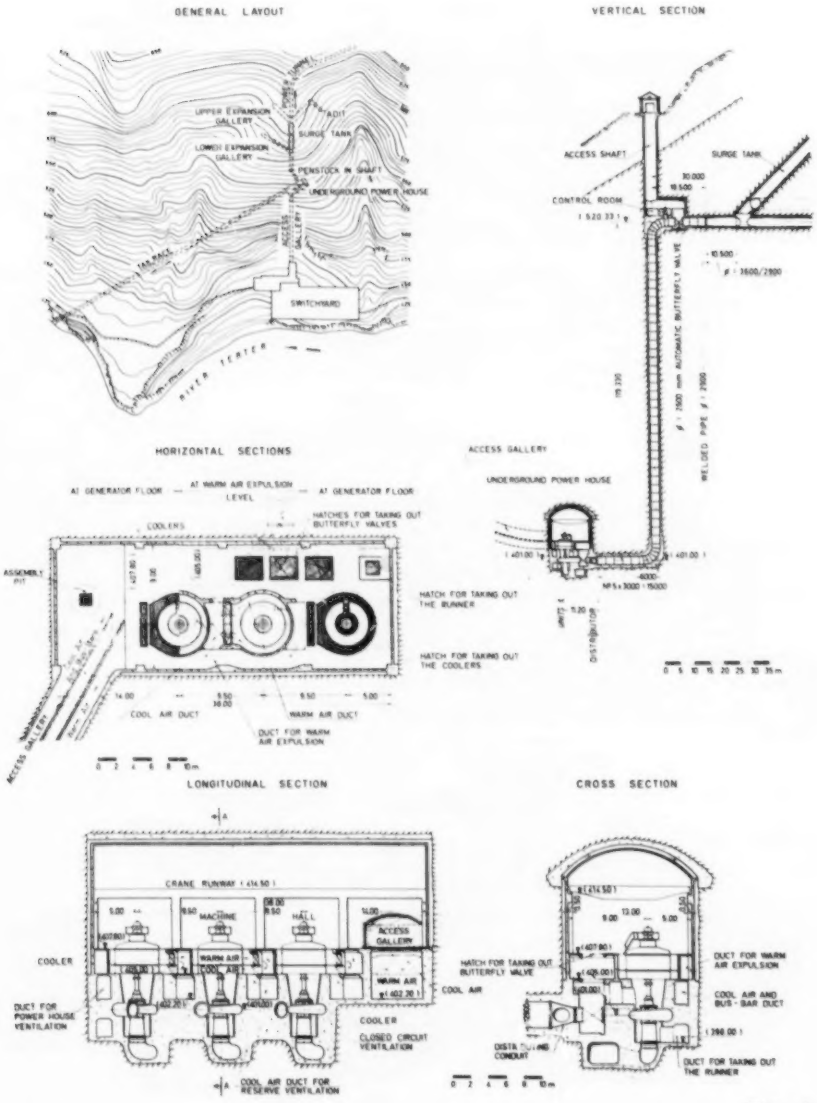
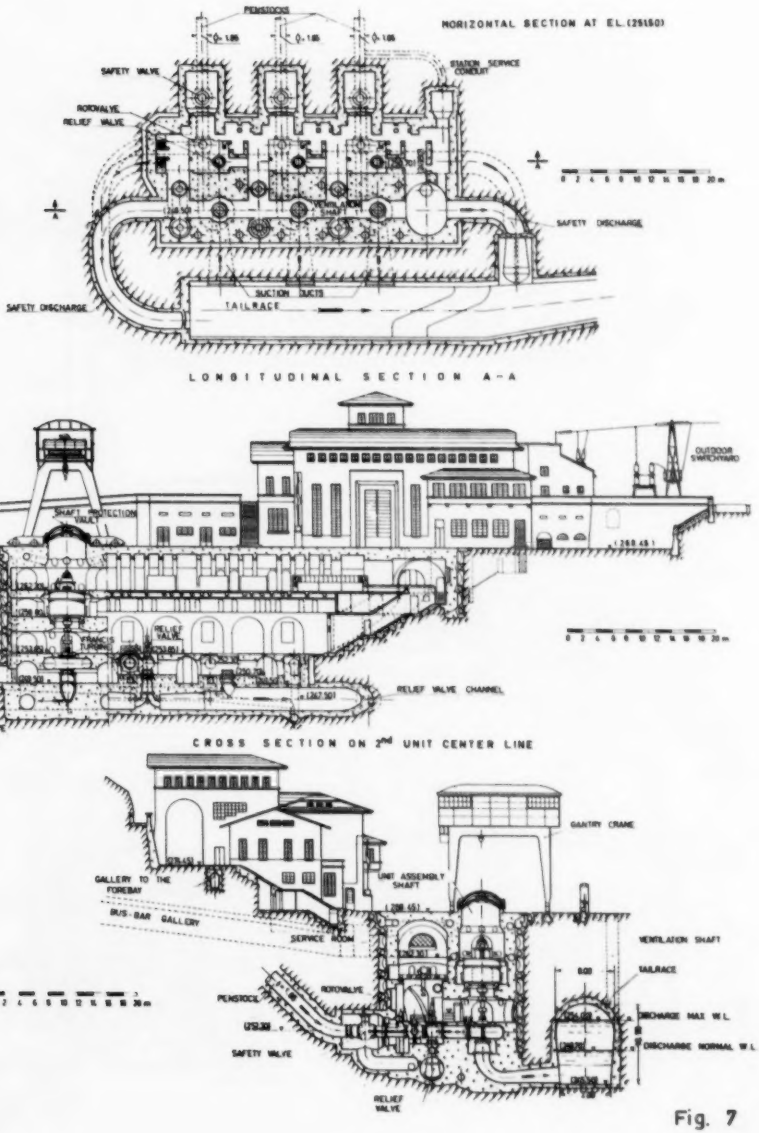


Fig. 6

CEDEGOLO - CIVIDATE HYDROELECTRIC PLANT

CIVIDATE POWER HOUSE

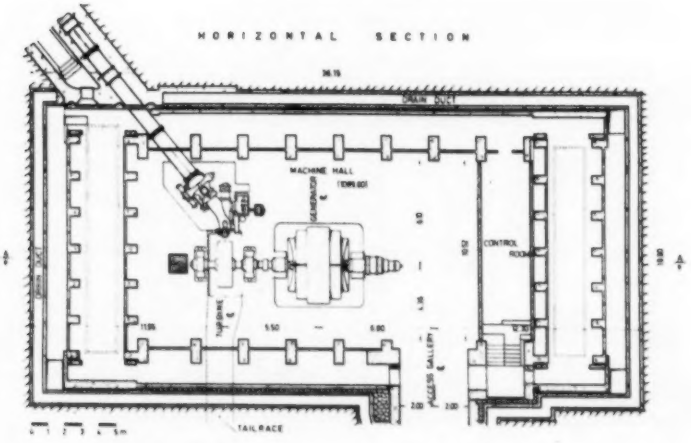


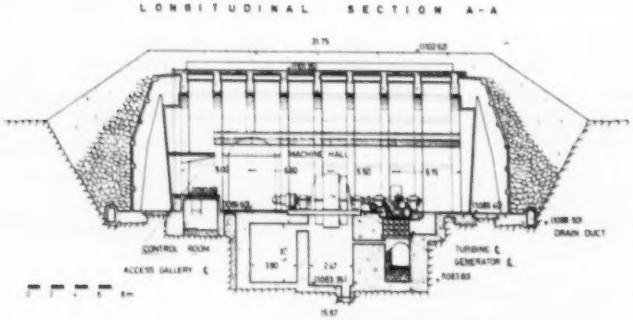
25320

Fig. 7

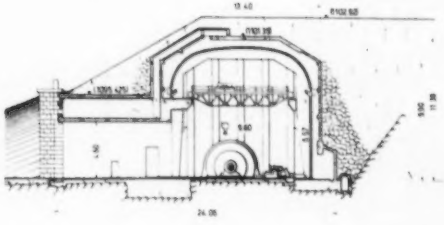
HYDROELECTRIC DEVELOPMENT OF THE RIVER BITTO-2nd PLANT

GEROLA POWER HOUSE





SECTION ON ACCESS GALLERY CENTER LINE



SECTION ON TURBINE CENTER LINE

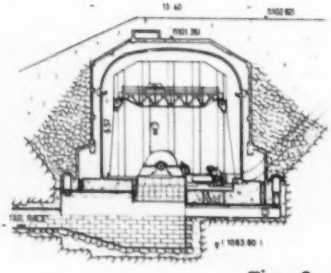
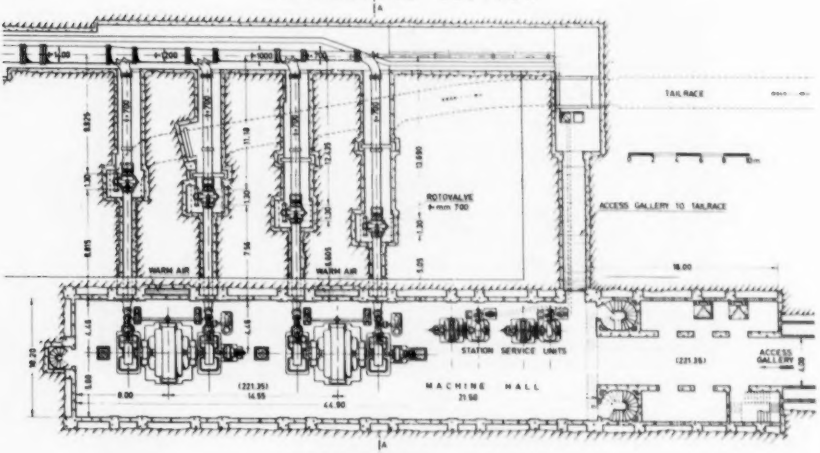


Fig. 8

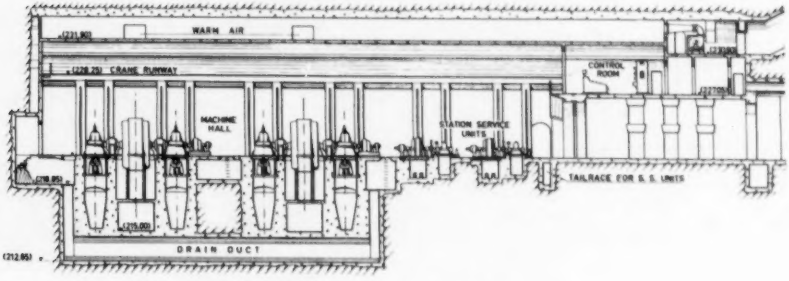
HYDROELECTRIC DEVELOPMENT OF THE RIVER BITTO-4th PLANT

REGOLEDO POWER HOUSE

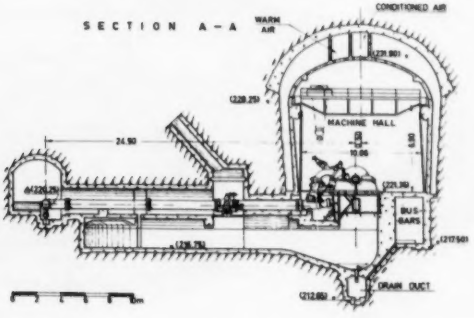
HORIZONTAL SECTION



LONGITUDINAL SECTION



SECTION A-A

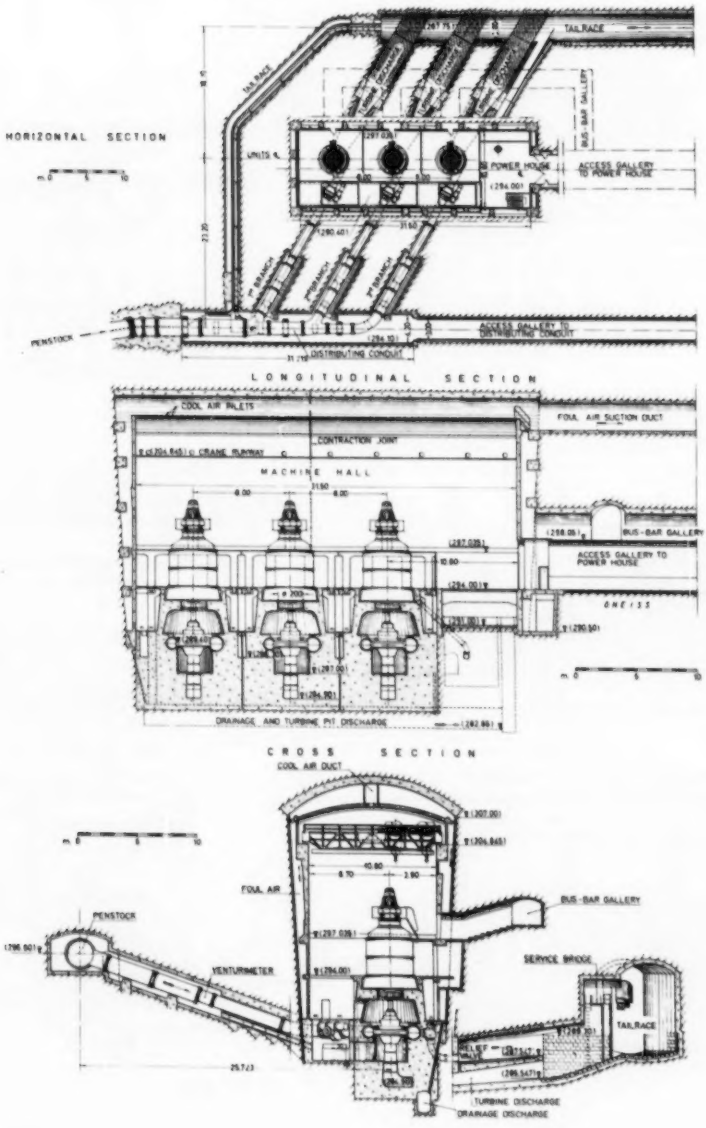


25326

Fig. 9

HYDROELECTRIC DEVELOPMENT OF THE RIVER MERA - 1° PLANT

PRATA POWER HOUSE



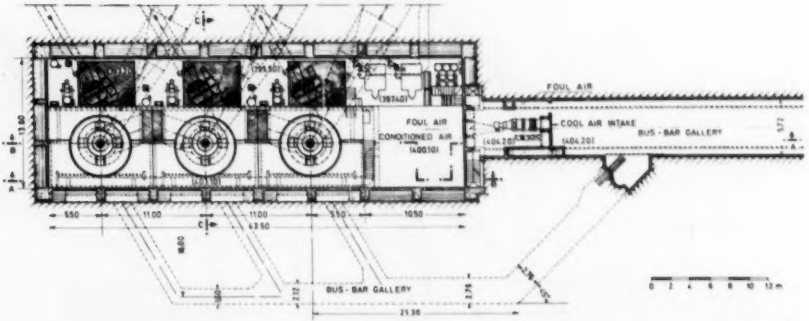
25329

Fig. 10

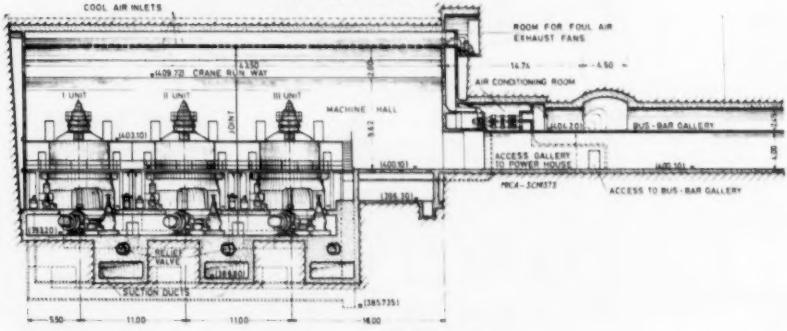
HYDROELECTRIC PLANT ON THE RIVER OGLIO

CEDEGOLO POWER HOUSE

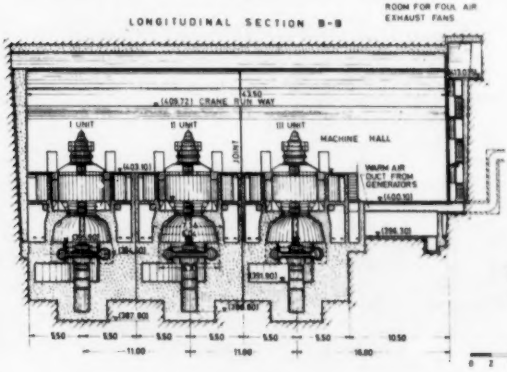
HORIZONTAL SECTION AT EL. (406.00)



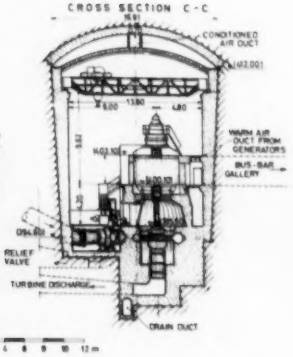
LONGITUDINAL SECTION A-A



LONGITUDINAL SECTION B-B



CROSS SECTION C-C



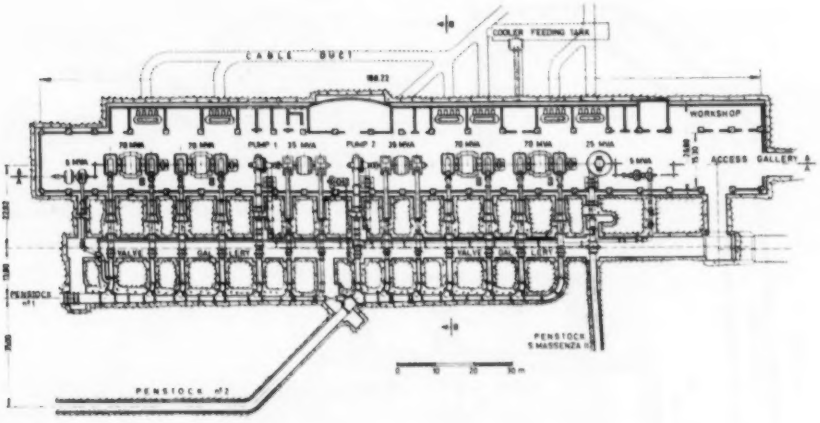
25330

Fig. 11

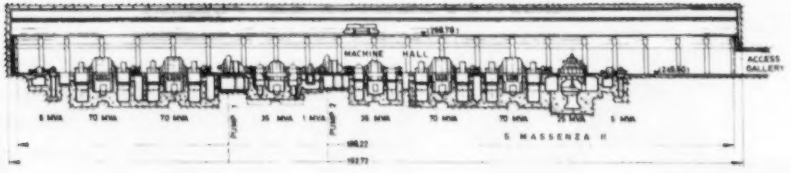
S. MASSENZA HYDROELECTRIC PLANT

POWER HOUSE

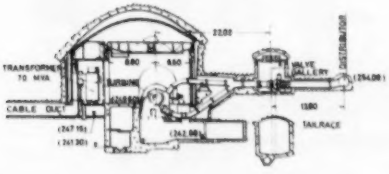
HORIZONTAL SECTION



LONGITUDINAL SECTION A-A



CROSS SECTION ON TURBINE IX AND ON 70 MVA TRANSFORMER



CROSS SECTION ON PUMP 2 AND ON CONTROL ROOM

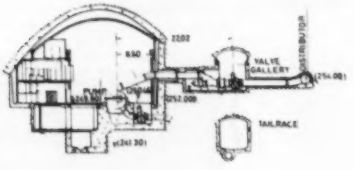
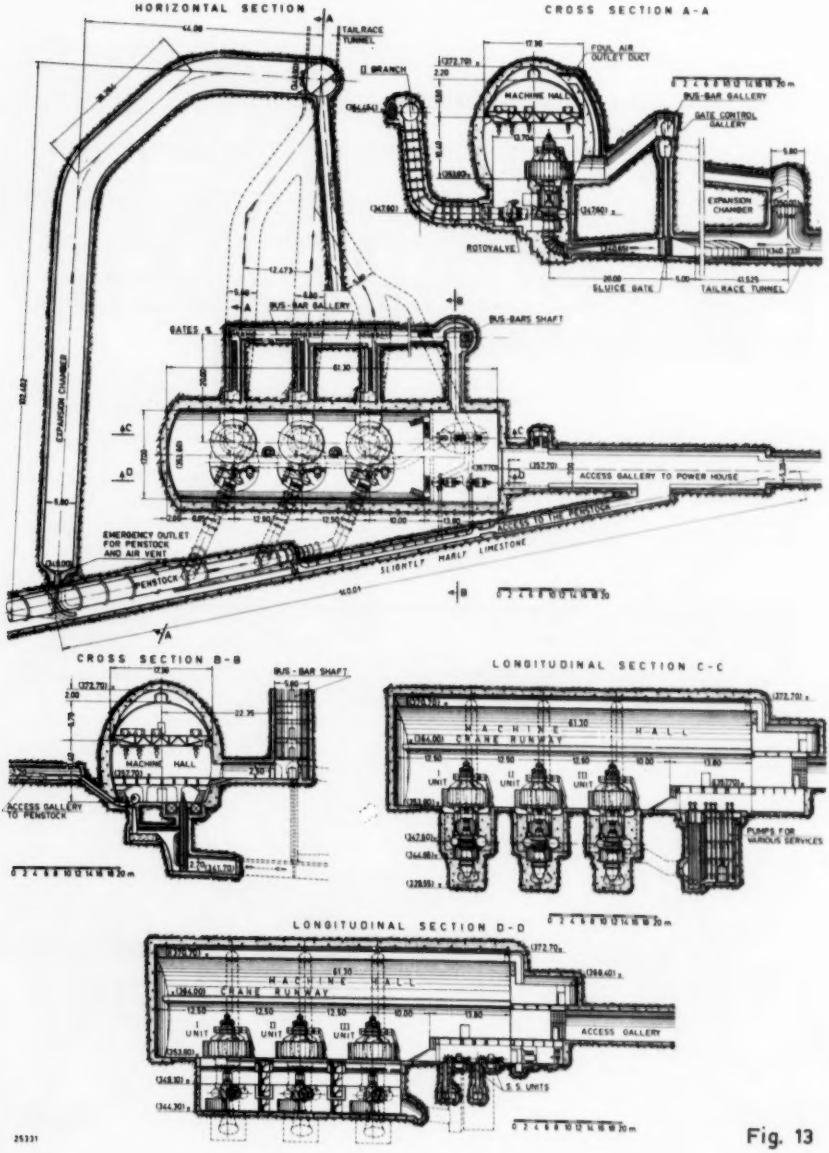


Fig. 12

HYDROELECTRIC PLANT ON THE RIVER NOCE FROM S. GIUSTINA TO MOLLARO

TAIO POWER HOUSE



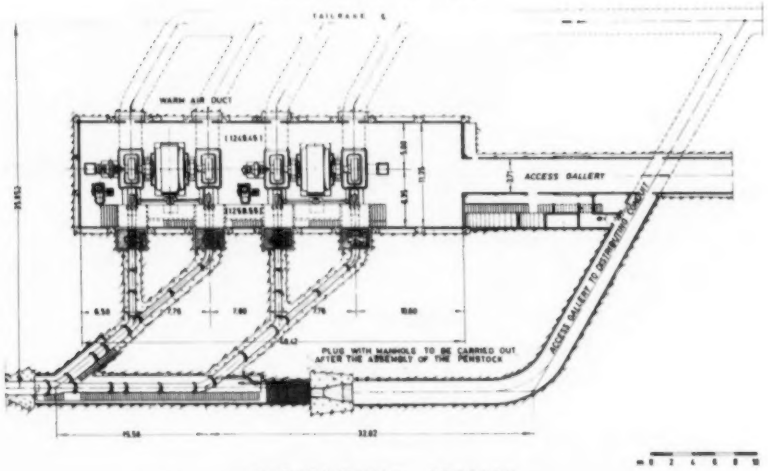
25331

Fig. 13

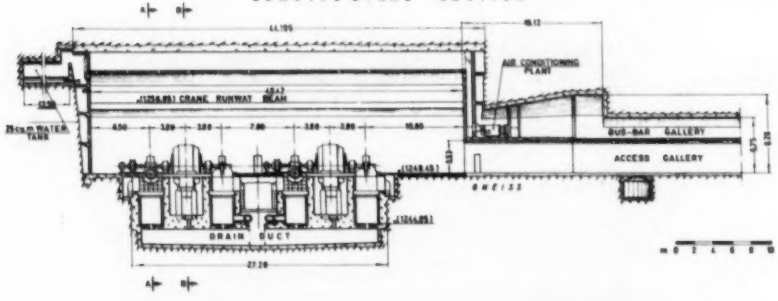
HYDROELECTRIC DEVELOPMENT OF THE RIVER LIRO - 1st PLANT

ISOLATO POWER HOUSE

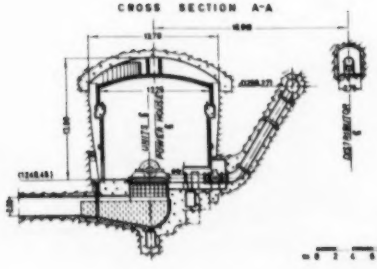
HORIZONTAL SECTION


TAILRACE
WARM AIR DUCT
ACCESS GALLERY
ACCESS GALLERY TO DERIVATION TUNNEL
PLUG WITH MANHOLE TO BE CARRIED OUT AFTER THE ASSEMBLY OF THE PENSTOCK

LONGITUDINAL SECTION



CROSS SECTION A-A



CROSS SECTION B-B

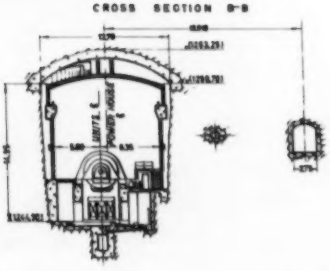
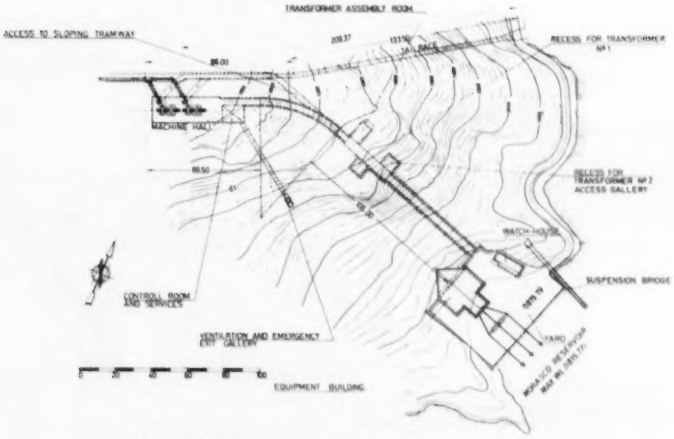


Fig. 14

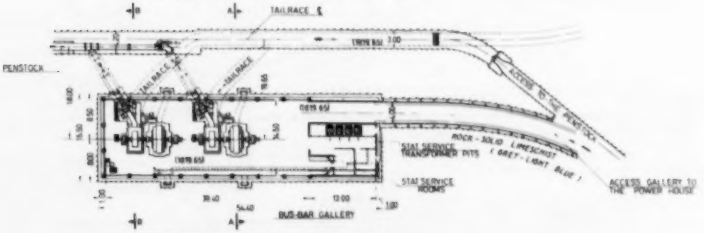
SABBIONE HYDROELECTRIC PLANT

ING. PIERO FERRERIO POWER HOUSE

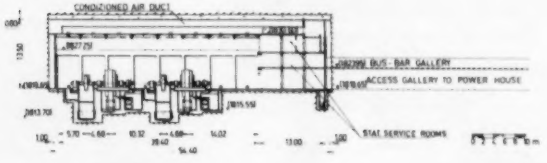
GENERAL LAYOUT



HORIZONTAL SECTION



LONGITUDINAL SECTION



25333

Fig. 15

PANTANO D'AVIO HYDROELECTRIC PLANT

POWER HOUSE

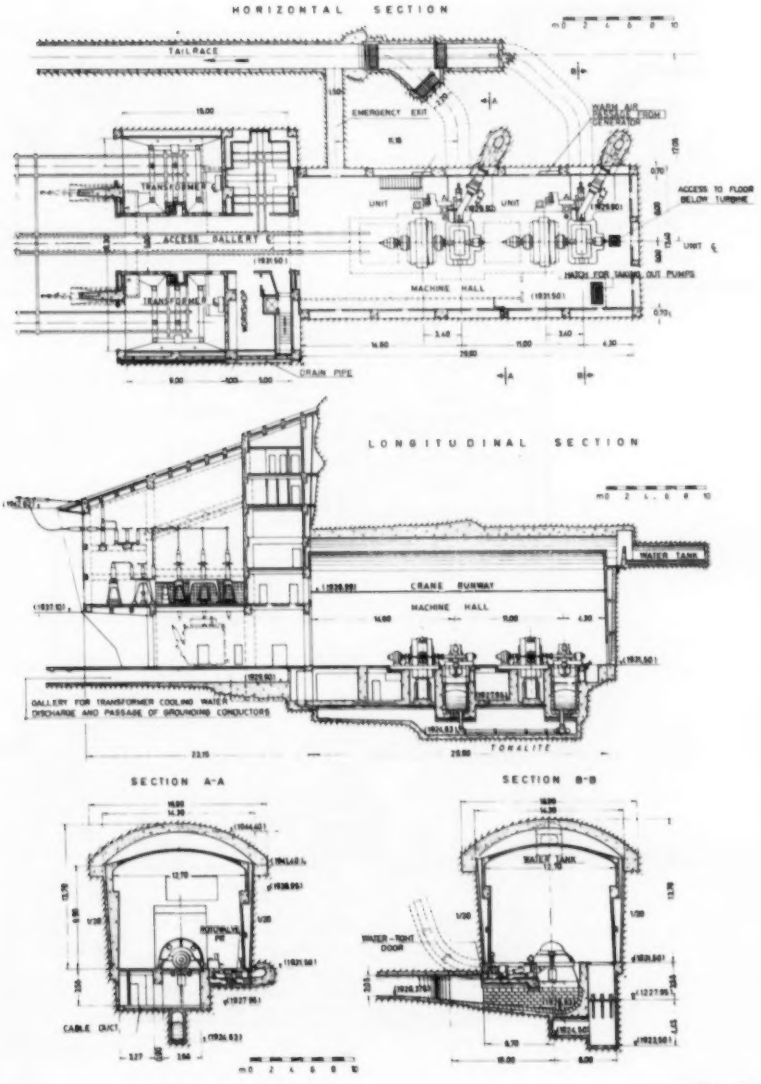
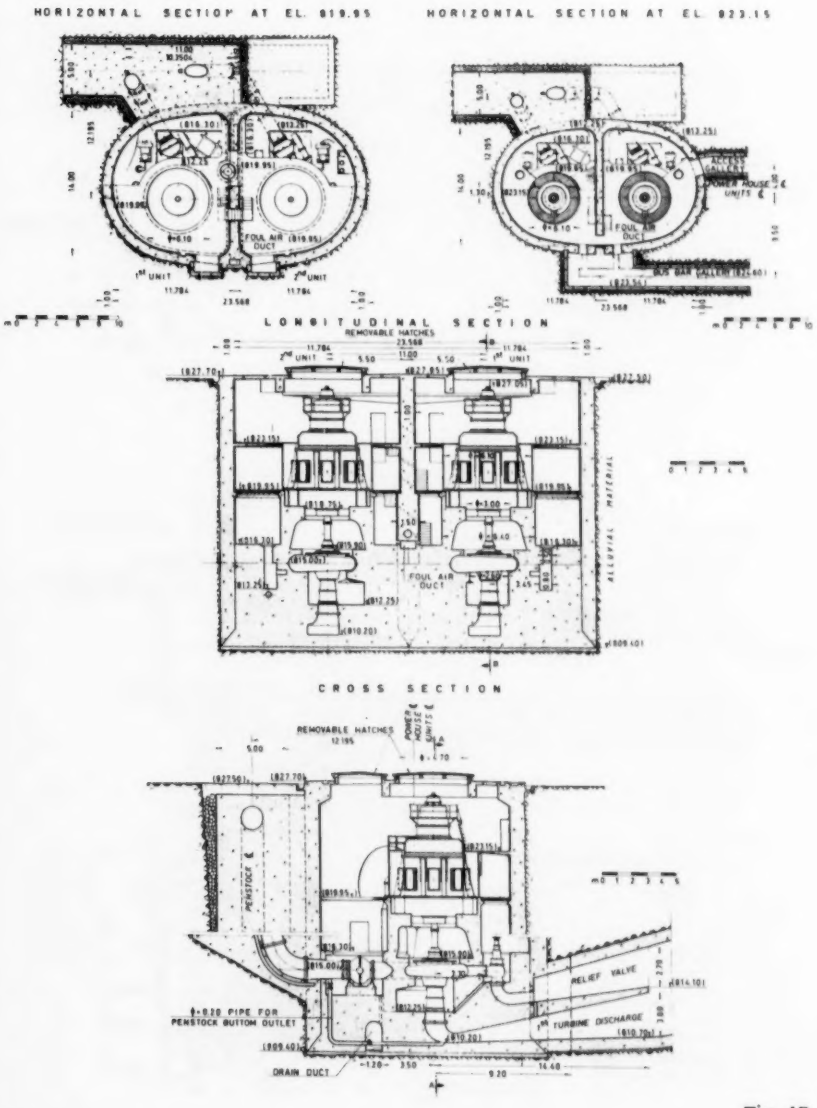


Fig. 16

HYDROELECTRIC DEVELOPMENT OF THE RIVER STURA DI DEMONTE-2nd PLANT

VINADIO POWER HOUSE



25336

Fig. 17

NEMBIA HYDROELECTRIC PLANT

POWER HOUSE

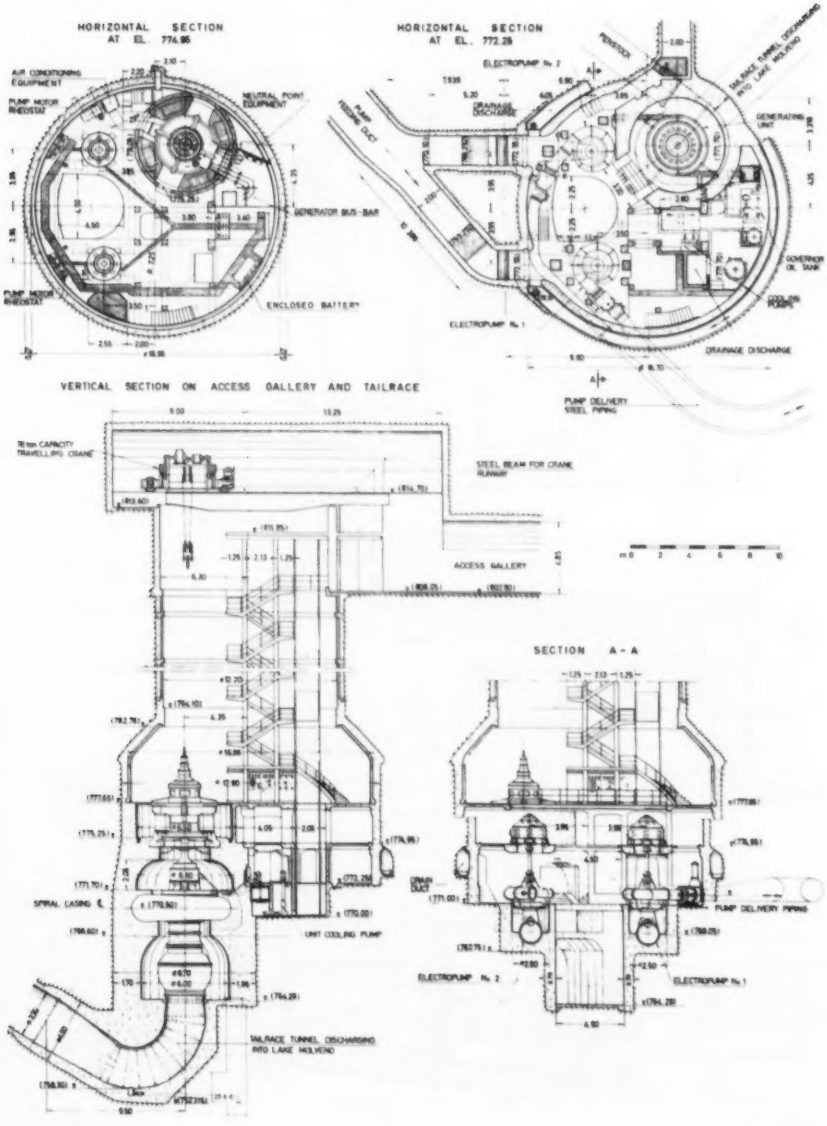
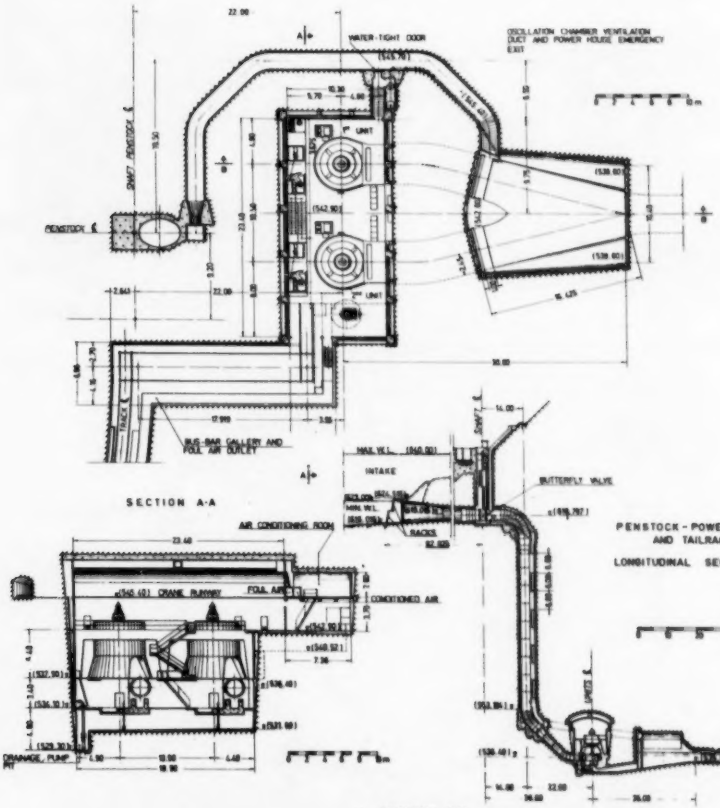


Fig. 18

HYDROELECTRIC DEVELOPMENT ON THE RIO JUQUIÀ - 1st PLANT (BRAZIL)

POWER HOUSE

HORIZONTAL SECTION



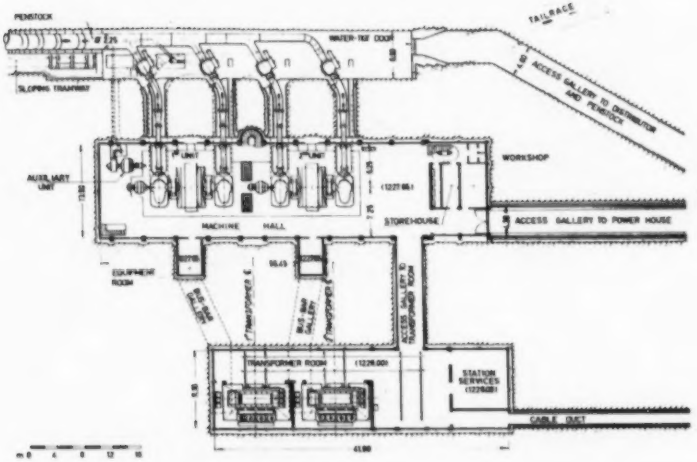
25237

Fig. 19

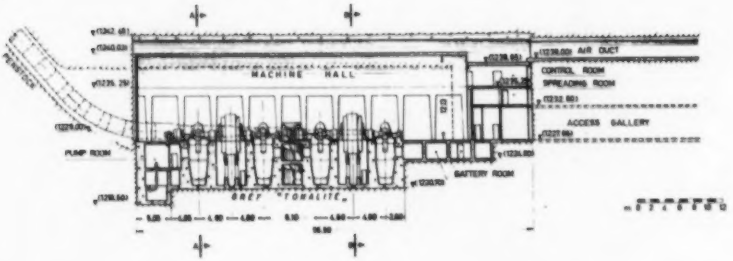
HYDROELECTRIC PLANT ON RIVER CHIESE (BISSINA - BOAZZO)

BOAZZO POWER HOUSE

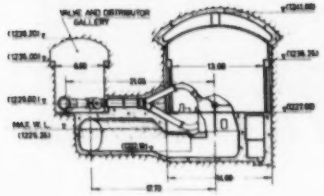
HORIZONTAL SECTION



LONGITUDINAL SECTION



SECTION A-A



SECTION B-B

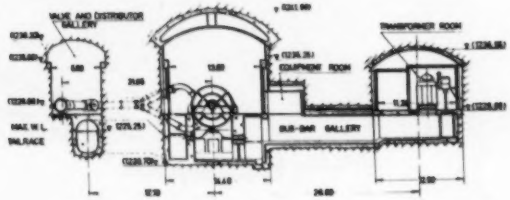
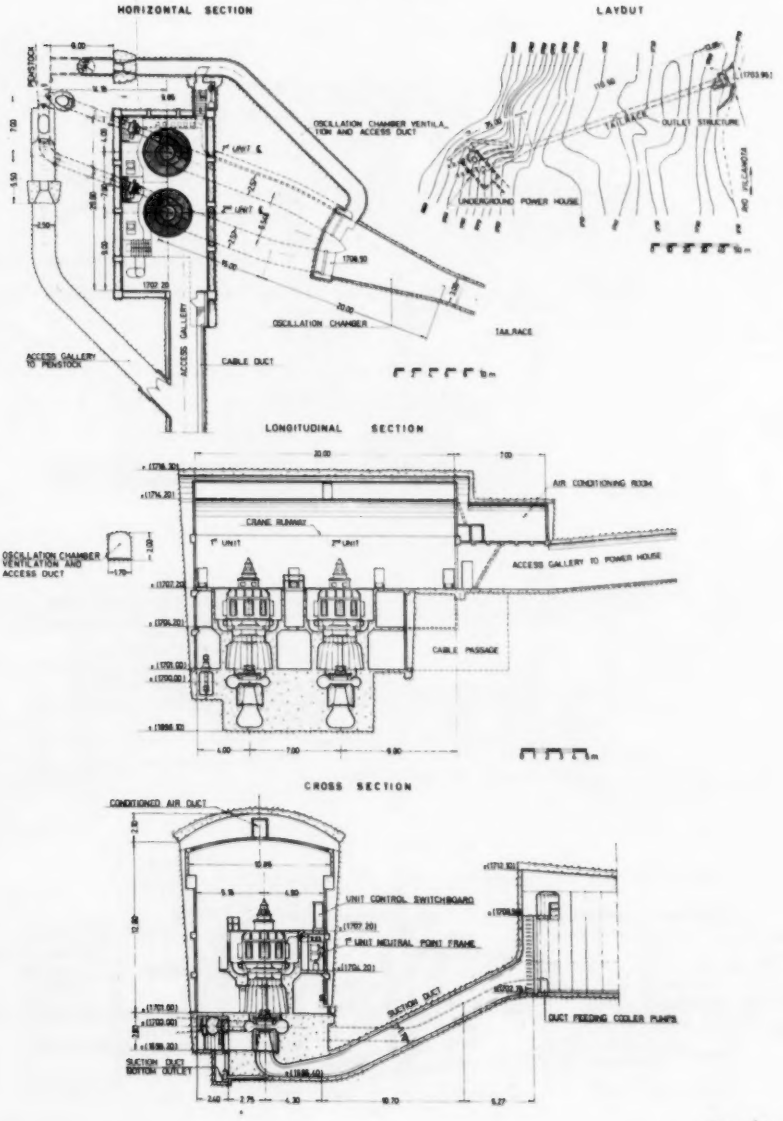


Fig. 20

MACCHUPICCHU HYDROELECTRIC PROJECT (PERÙ)

POWER HOUSE



25341

Fig. 21

Fig. 23 - Bitto No.4 Plant - Machine Hall

Fig. 25 - Mera No.1 Plant - Machine Hall

Fig. 26 - Santa Giustina Plant - Machine Hall

Fig.27 - Senico-Cedegolo Plant - Machine Hall

Fig.38 - Somico-Sedegolo Plant - Concreting of the retention wall during excavation of the cavern.

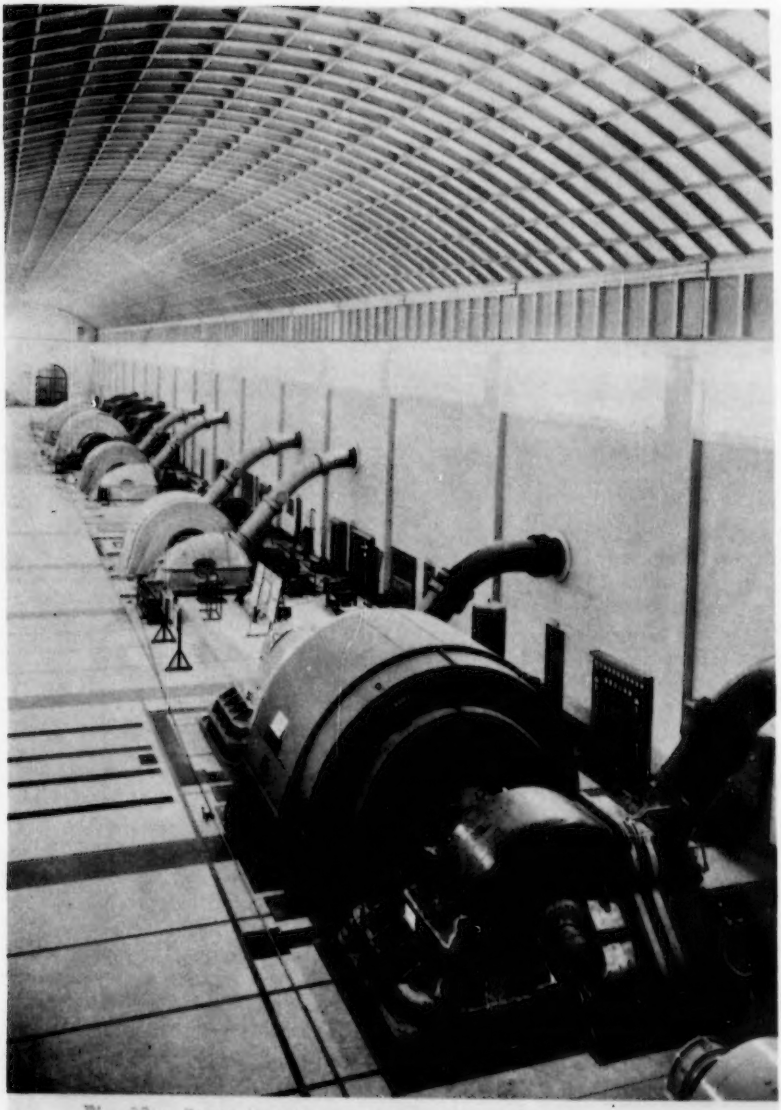


Fig. 29 - Santa Massenza Powerhouse - Machine Hall

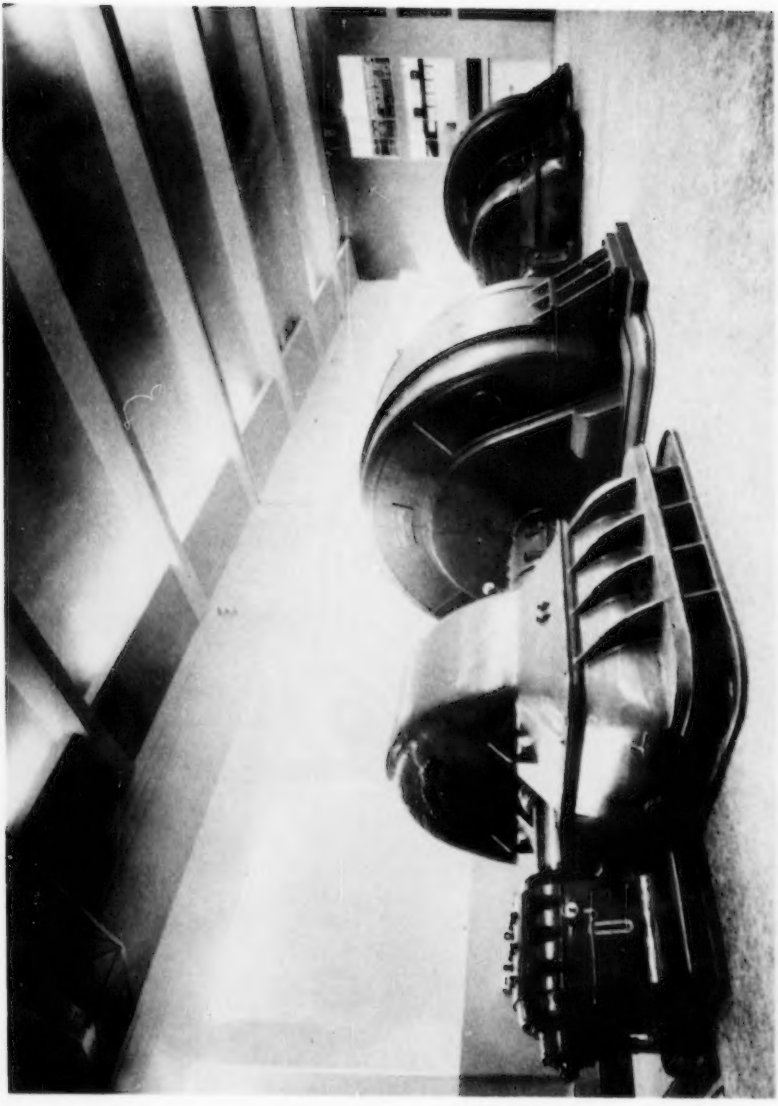


Fig. 30 - Cabbiana Unit - Machine Hall

Fig. 31 - Pantano d'vio Inst - Machine Hall

Fig. 32 - Aura Sea Plant - Transformer assembly building and access shaft

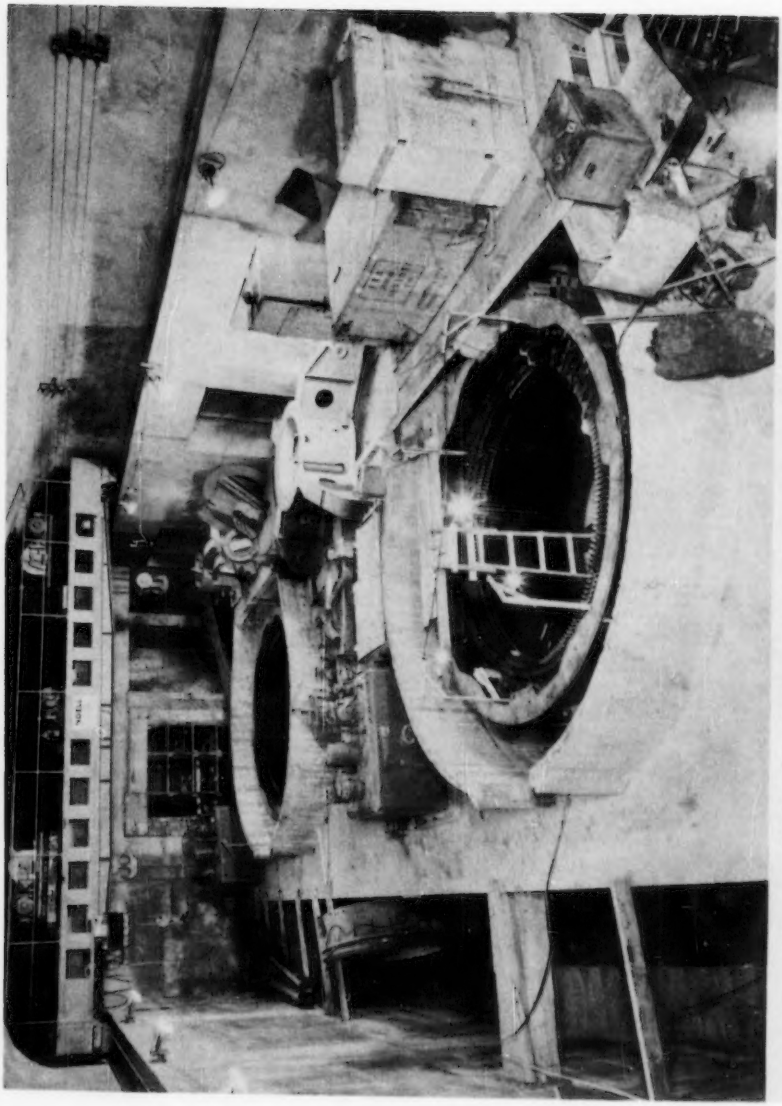


Fig. 33 - Jupiter Mod. Plant - Assembly of Generators

Paper 1555 February, 1958 PO 1

Journal of the
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Proceedings of the American Society of Civil Engineers

THE SUDAGAI UNDERGROUND POWER PLANT, JAPAN

Tatsuo Mizukoshi[1]
(Proc. Paper 1555)

ABSTRACT

A general description of the 46,000 kw Sudagai project is followed by more detailed information on the methods used in excavating and concreting the underground powerhouse. Costs of excavation and concrete are given. Model studies were made for the design of the tailrace surge chamber and particular attention was given to the question of moisture, temperature and lighting in the powerhouse. Seismic forces were considered in the design.

INTRODUCTION

In the snows of Mt. Tangoyama, 1,800 meters in altitude and located in the north-central part of Honshu Island, Japan, originates the Tone River, one of the largest in this country. The Tone flows in a generally southerly direction through relatively rugged country to emerge into the Kanto Plain which it crosses in a south-easterly direction to drain into the Pacific Ocean at Choshi.

The Sudagai Power Plant is situated at the confluence of the Naramata and Tone Rivers in the upper reaches of the latter. The construction work was started in 1952, completed in 1955 and the plant was immediately put in commerical operation. Fig. 1 shows the location of the Sudagai Power Plant in relation to the Tone Hydro-electric Development.

The Sudagai Dam is a gravity type concrete dam with a height of 75 meters. Water from the reservoir is led to the power plant which is constructed at the left side of the dam about 60 meters underground, where it supplies two 23,000 kw Francis turbines and is discharged into a tail race tunnel extending over 1,700 meters to join the Tone main stream again. In other words, the

Note: Discussion open until July 1, 1958. A postponement of this closing date can be obtained by writing to the ASCE Manager of Technical Publications. Paper 1555 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. PO 1, February, 1958.

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so-called "head-development type" was adopted in this plant. Therefore, a surge chamber is provided at a point between the power house and the tail race tunnel.

This development is the first large scale underground power plant worthy of notice in Japan. Some of the features of the work are described in this paper.

Outline of the Project

Principal project data are given in Table 1. Originally this plant was planned as the so-called "tail development type" having an intake, discharge pressure tunnel (1,700 meters long), surge tank, turbines and a short tail race. Geological surveys showed that the originally projected tunnel route was not adequate for building a pressure tunnel because the route lay through a serpentine formation and because suitable sites for the surge tank, penstocks and powerhouse were not available. After making comparative studies, the head developed type underground power plant was adopted for both economic and technical reasons.

TABLE 1 - GENERAL DESCRIPTION OF PROJECT

Dam	
Type	Crest overflow type concrete gravity dam
Height	75 m
Crest Length	194 m
Volume of Concrete	204,000 m^3
Reservoir	
Catchment Area	310 km^2
Effective Storage	22,000,000 m^3
Drawdown	25 m
Penstock	2 lines, dia. 3.70-2.65 m
Effective Head	73 m
Max. Water Discharge	65 m^3/sec.
Max. Power Output	46,000 kw
Turbine	2 - 23,000 kw - Francis type, 250 rpm
Generator	2 - 24,000 kva, 250 rpm
Power Plant	Underground type
Estimated Energy Output/Year	167,800,000 kwh

Conspicuous features in design and construction are as follows:

a. In construction of the underground power plant, thorough investigations were made of surging and air hammering of a surge chamber, ventilation, air temperature and moisture adjustment, underground vibration, etc. These questions were carefully studied and appropriate tests were performed.

b. Fly-ash was mixed with cement for dam construction as pozzolan with a view to improving the concrete quality and to reducing the construction cost.

c. Aggregate was quarried from liparite rocks near the plant site and processed by a series of crushing plants. This series of crushing plants had a capacity of 216 tons per hour.
d. Measuring instruments (strain gages, resistance thermometers, uplift meters, reinforcing bar stress meters, joint meters and photo-elastic stress measuring devices) numbering as many as 354 were embedded in the dam concrete for observation of the structural behavior of the dam. This observation is expected to be continued for some time to come.
e. The two coffer dams were built as arch dams. One of these two dams, the Naramata coffer dam, 12.8 meters in height and 72.4 meters in length, was used for various experimental purposes (such as experiments in temperature changes in the dam concrete, dam stresses and deformations, solid bed rock deformation after loading, dam characteristics against vibration, etc.)
f. For the purpose of discharging warmer water from the reservoir for the use of irrigation, two intake structures were constructed on the upstream side of the dam to allow the discharge of warmer surface water from the reservoir despite fluctuations in the reservoir water level.

Structure of the Underground Power Plant

General features of the underground power plant are shown in Figs. 2 to 4. Water is taken from the two intake structures at the upstream side of the dam and through the two embedded penstocks, vertically about 50 meters and horizontally about 60 meters long, led into the two turbines and then through the draft tubes discharged downwards into the afterbay to be carried into a tail race tunnel.

The power plant is in two parts, the underground and the above-ground portions. Under the ground are turbines, generators, drain pumps, butterfly valves, cubicle room and ventilator room. Above the ground is a reinforced concrete building containing the switchboard, battery room, house service transformer room, transfer bus room and service office. The two portions are connected with an elevator. For carrying heavy equipment into the cavern and out again an inclined access tunnel is used. The general description of the underground power house and the surge chamber is given in Table 2.

TABLE 2 - GENERAL DESCRIPTION OF UNDERGROUND POWERHOUSE AND SURGE CHAMBER

	Underground Powerhouse		Surge Chamber
	Generator Room	Accessory Room	
Length	35,400 m	20,700 m	61,200 m
Height	31,003 m	12,553 m	24,900 m
Width	16,600 m	15,850 m	6,200 m
Volume of Excavation	21,533 m^3		7,897 m^3
Concrete Volume	7,730 m^3		2,364 m^3

Design of the Power Plant

Geological and petrological investigations showed the poser plant site to consist of coarse grained granite. The points to be taken into consideration in designing were:

a. The prevention of the rock falls in the cavern.
b. The stabilization of the rock structure against ground pressure and seismic force and
c. The influence of pressure of the underground water permeating from the reservoir.

An underground cave of this scale excavated in a massive rock of monolithic quality is generally thought to be stable against ground pressure without having concrete lining. Therefore, an egg shaped section was not adopted in this case but in consideration of joints and fissures in the rock, concrete lining is thought to be necessary to prevent rock falling and also of advantage for maintenance of the cave in the future.

Regarding the difference of seismic forces above the ground surface and under the ground, the records that had been kept in Japan show that the subterranean seismic force is smaller than that on the ground surface, but no definite conclusion was reached by the present investigations.

Regarding the influence of pressure of underground water permeating from the reservoir, it was deduced that the influence would be minimized by grouting the rock around the powerhouse and also drilling a curtain of drain holes.

In view of the above, the following principles were established:

a. As regards the loads acting on the arched roof of the cave, a uniformly distributed load equal to the weight of a rock mass of an inverted "U" shape with a height of one half of the cave width was taken in consideration of the separation of the arch concrete from the rock ceiling after lining and the falling of the loosened rock layer. No consideration was taken of the underground water pressure and the secondary stresses caused by temperature changes.
b. The shape of the arched roof was determined from the economic viewpoint. By comparing curves of various shapes a parabolic curve line was adopted and the thickness of the lining and the reinforcement of the cave were determined accordingly.
c. In the design of the side wall, the horizontal component of ground pressure acting on the vertical concrete wall was not taken into consideration but the side rock behind the concrete wall was firmly bolted and grouted. The thickness of the concrete wall and the amount of the reinforcement were determined taking only the seismic force against the cantilever wall (equivalent to the horizontal seismic coefficient of 0.12) into consideration.
d. To minimize the influence of the pressure of the underground water permeating from the reservoir, it was decided that the powerhouse would be surrounded by a drainage tunnel, from which drain holes would be vertically drilled upward; in other words the powerhouse would be surrounded with a curtain of drain holes.
e. To keep out the influence of moisture and temperature deriving from the surrounding rock, a second wall was added, thereby creating an interspace between the first and the second walls. In addition a ventilation system was provided.

General Geology

The nature of the rock right above the underground power plant was made clear by test borings.

According to the report by the Technical Research Institute of Electric Power, the nature of rock samples picked up in the underground cave, when analyzed, was as follows:

a. The rock is coarse grained granite but somewhat jointed and weathered. Quartz in this granite is rose colored and felspar is kaolinized as white powder and biotite is changed into chlorite.
b. Analyzing 9-14 cm cubic test pieces made from rock samples taken underground, the physical properties of the rock were found to be as shown in Table 3.

TABLE 3 - NATURE OF THE ROCK

No.of Test Piece	Specific Gravity	Absorption %	Compressive Strength (kg/cm^2)	Remarks
1	2,598	0.49	515	Remarkably weathered and much jointed
2	2,623	0.15	1,231	
3	2,611	0.22	1,173	
4	2,602	0.43	684	Considerably weathered, much jointed and unevenly loaded
5	2,614	0.24	1,074	Badly decomposed and jointed
6	2,550	0.98	581	
7	2,612	0.39	1,023	
8	2,617	0.31	1,070	
9	2,603	0.39	718	From uneven formation of the test piece, the rock was concluded as unevenly loaded.

Excavating and Concreting Work

As to the method of excavation, in addition to the vertical shaft for the powerhouse and an inclined access tunnel both of which became permanent facilities, two inclined adits for conducting excavation and one adit for concrete placing were dug. The surge chamber was also excavated by means of one inclined drift and one adit from the tail race. The excavation of the

turbine room and the penstock tunnel was conducted by means of the same adit from the tail race.

The underground powerhouse having a volume of approximately 21,530 cubic meters was divided into four zones, A, B, C and D, and a kind of bench method was adopted. Data of each zone to be excavated by the above four inclined drifts are shown in Table 4.

TABLE 4 - DESCRIPTION OF THE INCLINED DRIFTS USED FOR EXCAVATION OF UNDERGROUND POWERHOUSE

Name of Zone	Amount of Excavation	Name of Drift	Section	Lenght of Drift	Gradient
A	3,422 m^3	No.1 Inclined Drift	4.0x2.5m	45.940m	1:3.5
B	6,700	No.2 Inclined Drift	4.0x2.5	47.066	1:?.6
C	5,522	Access Tunnel	4.0x2.5	81.286	1:3.0
D	5,889	Tail Race Inclined Drift	4.0x2.5	143.955	1:3.45
Total	21,533	-	-	-	-

No. 1 inclined drift (A zone) branched into three headings, after reaching EL 676.40 meters, two of which were excavated towards both walls and one of which was towards the top of the arch crown. Then these headings were connected together at No. 1 Unit side. The connection of the three headings was enlarged towards No. 2 Unit side, with the central part of A zone rock being left for convenience of steel forms erection for arch concreting. After removal of the arch steel forms the central part of A zone rock was excavated. No. 2 inclined drift (B zone) and access tunnel (C zone) also branched into three headings respectively and each zone was enlarged from these headings. The tail race inclined drift headed for the afterbay and was divided into two headings at the end. Each heading headed for the penstock tunnels through the surge chamber and the draft tubes. These headings enlarged the afterbay, the lower part of the surge chamber, draft tubes, D zone and horizontal penstock tunnels. The ratios of heading excavation versus enlargement were 14% in A zone, 13% in B zone, 19% in C zone and 17% in D zone.

Rock bolting methods were often employed in excavation to prevent rocks from falling. The powder used for excavation was dynamite containing 28 to 32 per cent of nitroglycerine. Dynamite required per cubic meter of excavated rock was as follows:

A, B and C zones	Dynamite 1.87 kg/m^3
D zone	Dynamite 1.63 kg/m^3

Concrete was produced at the mixing plant having 2.8 cubic feet and 15 cubic feet mixers at the opposite side to the power plant and was conveyed by a 16.8 meters long transit cable to a hopper installed at the power plant side. From the hopper it was loaded on steel trolleys having 0.2 cubic meters capacity. Brought down along the working drift (50 meters long, 1:5 inclined)

concrete was placed on the arched roof first. In placing concrete on the arched roof, 11 arched steel forms, each of which was made so as to break up into 9 pieces, were used, being set at intervals of 1.2 meters. In places other than the arched roof, concrete was placed by means of concrete pumps installed just below the hopper. It took an average of 7 minutes to finish the concrete placing after concrete was produced at the batcher plant. Forms were removed at least 5 days after concrete was placed.

The unit cost of excavation and concrete of each part in the underground power plant is shown in Table 5.

TABLE 5 - UNIT COSTS OF EXCAVATION AND CONCRETE PLACING

	Unit Cost of Excavation Per Cubic Meter	Unit Cost of Concreting/Cubic Meter	
		Cement 300 kg/m^3	Cement 210 kg/m^3
Underground Cave	7,650 yen ($ 21.25)	9,420 yen ($ 26.17)	8,090 yen ($ 22.47)
Surge Chamber	7,000 yen ($ 19.44)	10,520 yen ($ 29.22)	8,560 yen ($ 26.56)
Inclined Drift & Access Tunnel	7,800 yen ($ 21.67)		

Note: In the above table the conversion rate of 360 yen to 1 Dollar was used.

Surge Chamber

The tail race tunnel of this power plant is of horseshoe shape and normally free surface flow, but it becomes a pressure tunnel when the water level in the pondage of the Minakami Power Plant (located downstream) is raised. To relieve the tunnel of the water hammer produced by a sudden load change, a surge chamber was constructed between the powerhouse and the tail race tunnel. Dimensions of the surge chamber are shown in Figs. 2 and 3. Its design was based on model tests of the variable flow in the tail race and of the actual surging phenomena which substantially agreed with the expected behavior by theory. Furthermore, the model test showed that air inhaled into the tunnel during a down-surge was pressed by an up-surge and that a hammering action occured when the air was released from the tunnel into the surge chamber. The instantaneous pressure rise due to the above air release amounted to 20-25 meters, while the pressure drop was 4-5 meters, thereby changing the water head. To reduce the air hammering, a partition wall was made between the tunnel and the surge chamber (see Fig. 3), and an air pipe was installed connecting the tunnel to the surge chamber through the partition wall so that the compressed air might be released through this pipe to the atmosphere, not directly being released to the water surface in the surge chamber. The air in the tunnel does not seem to change pressure suddenly. As the result of this, no remarkable effect is produced by the air hammering action in the chamber.

Other Special Features

In designing the underground power plant, special consideration was given to the treatment of leakage, elimination of moisture, ventilation, method of lighting and prevention of noise.

Above the generator room floor, complicated double walls were constructed and the second wall added was independent of the rock wall in the cave.

To prevent leakage, the ground around the powerhouse was grouted and the powerhouse was further surrounded with a curtain of drain holes. To regulate the moisture and temperature, a second wall was built so as to have an interspace between the first and second walls. By this interspace, the powerhouse was kept from the moisture and temperature deriving from the surrounding rock. The ventilation facilities are sufficient to vent 4.3. times the room capacity of 8,845 cubic meters per hour and the air ducts were arranged in the interspace between the first and second walls. Much thought was given to securing an adequate temperature and humidity—viz. 25.5° C (77.9° F) in temperature and 60 per cent or less in moisture during summer and 15° C (59° F) and 60 per cent or less during winter. In order to produce a soft lighting effect, color conditioned fluorescent lighting was adopted and the intensity of illumination in each room is shown in Table 6.

TABLE 6 - INTENSITY OF ILLUMINATION

Rooms	Intensity of Illumination Mean Value	Remarks
Office Room	405 lux	
Switch Board Room		
Horizontal Mean	315	80 cm above the floor
On Operator's Desk	310	
On Operation Board	385	
Main Panel, Vertically	130	
Sub Panel, Vertically	110	1.5 m above the floor
Water Level Watching Board	115	
Emergency Lighting	15.6	80 cm above the floor
Generator Room	83	Measured on the floor
Turbine Room	32)	
Communication Room	258)	
House Transformer Room	102)	
Transfer Bus Room	64)	
Cable Room	30)	80 cm above the floor
Battery Room	26)	
Switch Room	30)	
Drain Pump Room	30)	
Ventilating Room	156)	
In Elevator	42	Measured on the floor
Wash Room	220)	
Toilet	104)	
Passage Way (1st Floor)	100)	80 cm above the floor
(3rd Floor)	120)	
Incline (upper	25)	
(inner)	3)	
Inspection Gallery	20)	Measured on the floor
Around Switch Yard	10)	
Access Road	5)	

In order to make the underground chamber soundproof, the inside wall was lined with acoustic slate and the rear wall was covered with rock wool. The arched roof was also lined with acoustic aluminum tile and covered with rock wool as was the rear wall. The noise measured at various parts of the underground chamber is as stated below:

TABLE 7 - NOISE MEASURED AT VARIOUS PARTS OF UNDERGROUND CHAMBER

Rooms	PHON*	REMARKS
Generator Room	80-81	One unit operated
Ventilator Room	94-96	
Turbine Room	91-93	
Control Board Room	90-95.5	
Pump Room	100-101.3	
Switch Board Room	45	
In Elevator	53	When operated
Cooling Machine Room	78	
Cubicle HG room	80	
Diesel Room	105	
Drain Pit	77	
Incline Operated	69-73	

* The PHON is a unit of "loudness" related to the decibel in accordance with the Fletcher and Munson curves.

Fig. 5 shows the construction of the second wall in the underground powerhouse.

Views of the dam and powerhouse are shown in Figs. 6-9.

Supplementary Remarks

In designing this underground powerhouse, the difference of seismic forces on the ground and under the ground was rather intensively studied. However, no definite conclusion was reached since the same seismic coefficients either on the ground or under the ground were obtained. According to the records made elsewhere by Japanese seismologists, the underground amplitude or displacement is less than that on the surface of ground. The ratio of displacement under the ground to that on the surface is, according to the records kept by Japanese seismologists, 0.25 and above but not over 1.0. This ratio, however, is dependent chiefly upon the period of an earthquake.

For the purpose of studying the subject more thoroughly, the Tokyo Electric Power Co. obtained the services of Dr. Okamoto who installed two Ishimoto type seismographs, one in the underground powerhouse and the other on the surface right above it and since early in 1956 the difference in amplitudes have been measured. This investigation has proved that the seismic force under the ground is smaller than that above the ground. For example, in one case when the period of an earthquake was 0.23 seconds, the ratio was shown to be 0.36. These studies are now nearing completion.

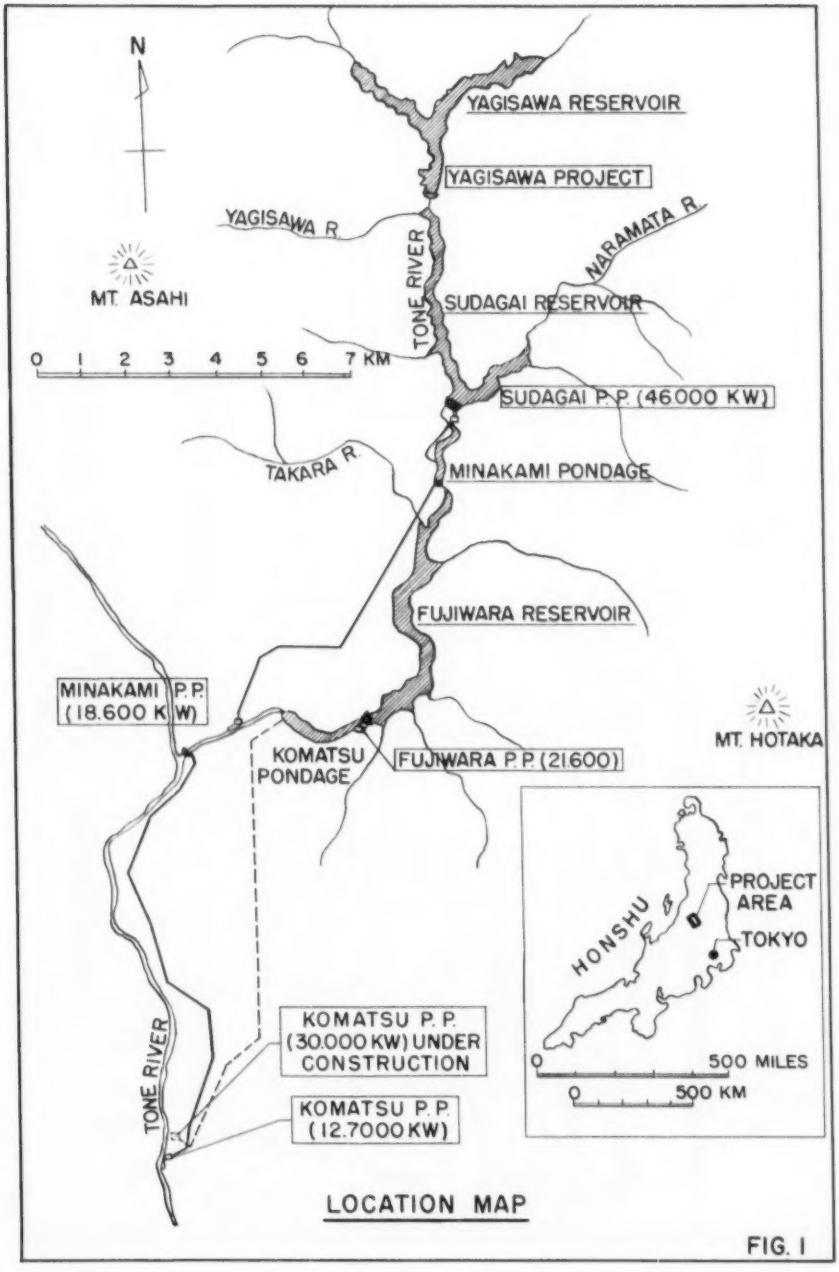
As regards the leakage in the drainage tunnel, it was expected that this would increase after filling the reservoir, because of the percolation of water from the reservoir through the drainage tunnel. The actual measurement, however, has shown that the same amount of leakage, 7 to 8 liters per minute, continued even after filling the reservoir and no considerable increase in leakage has been recognized up to the present time.

CONCLUSION

The experience gained through the construction of this power plant was valuable to us particularly in studies of civil engineering. This plant is the first complete underground power plant Japan has ever constructed. For this reason, based upon various kinds of model tests, the design and the construction were carried out with great care. One thing that we can confidently affirm is that if geological conditions had been more adequate, simplified structure would have been possible with resultant reduction in construction cost.

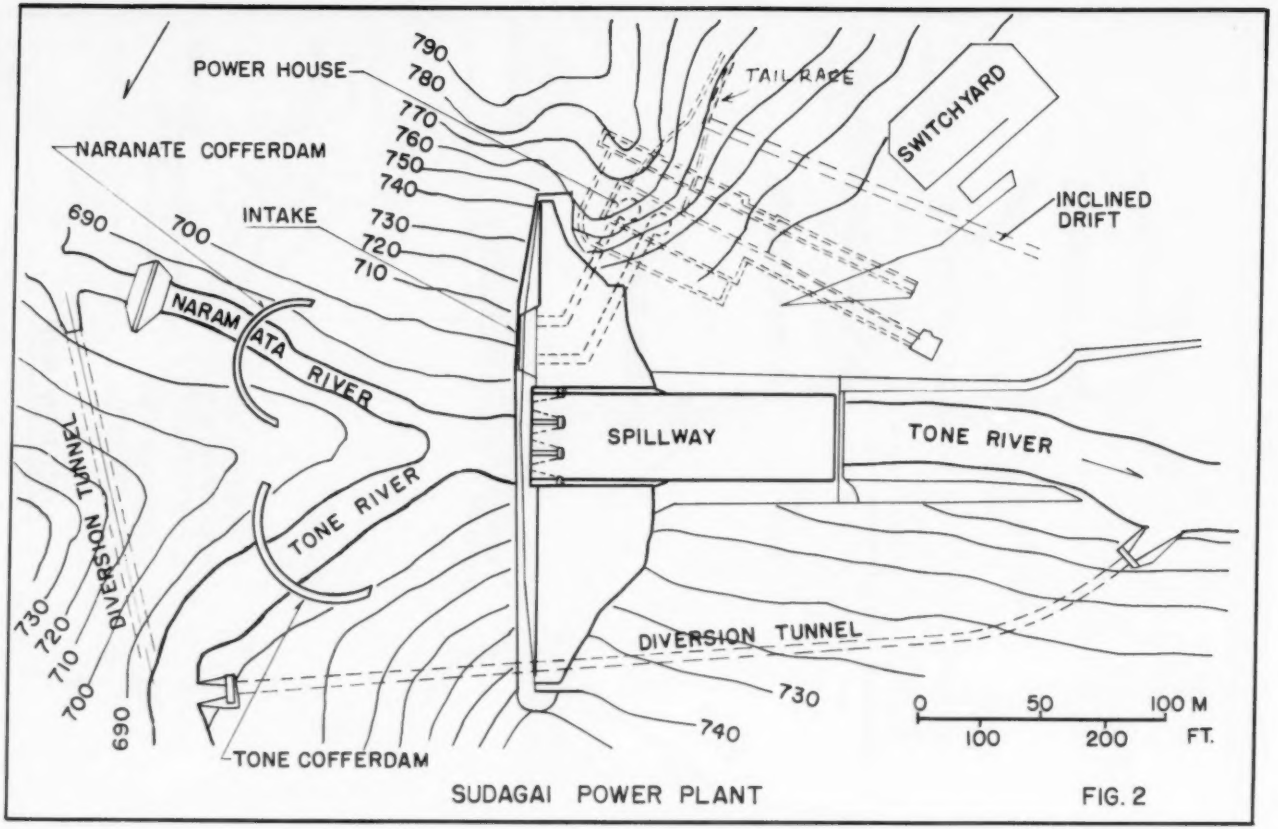
ACKNOWLEDGMENTS

The author is deeply indebted to the Tokyo University and the Technical Research Institute of Electric Power who rendered valuable advice and assistance for engineering and designing, and to Hazamagumi & Co., Ltd., Contractors of the project, who faithfully executed the work. The author takes this opportunity to express his appreciation to them for their willing cooperation in carrying out this project.



LOCATION MAP

FIG. 1



SUDAGAI POWER PLANT FIG. 2

PROFILE OF POWER HOUSE & SURGE CHAMBER

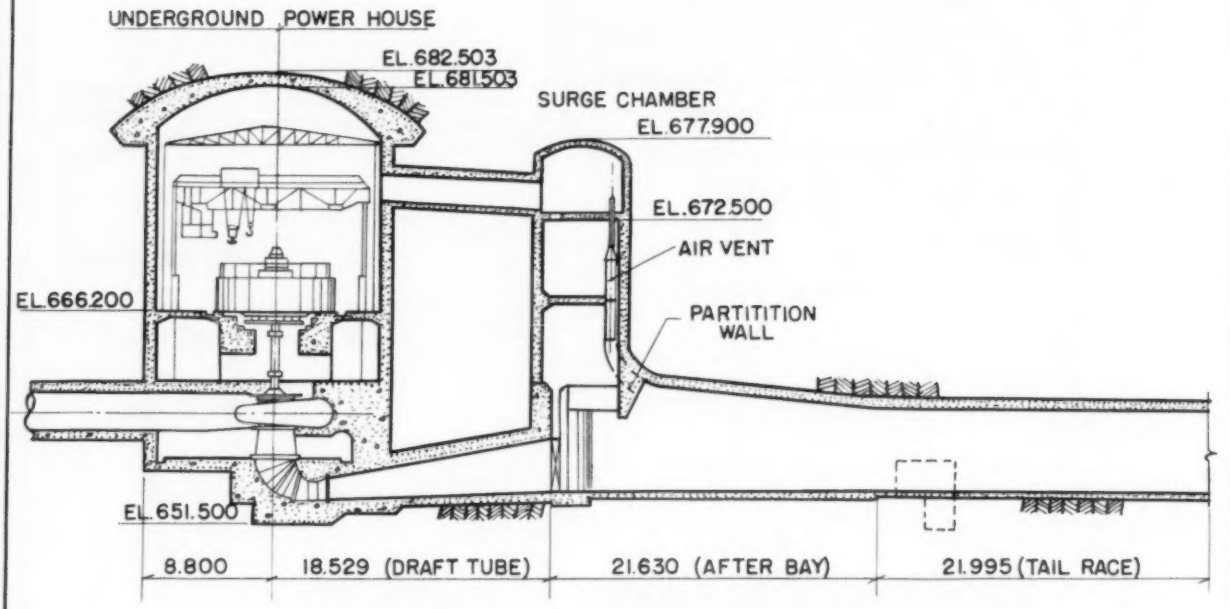


FIG 3

CROSS SECTION OF UNDERGROUND POWER HOUSE

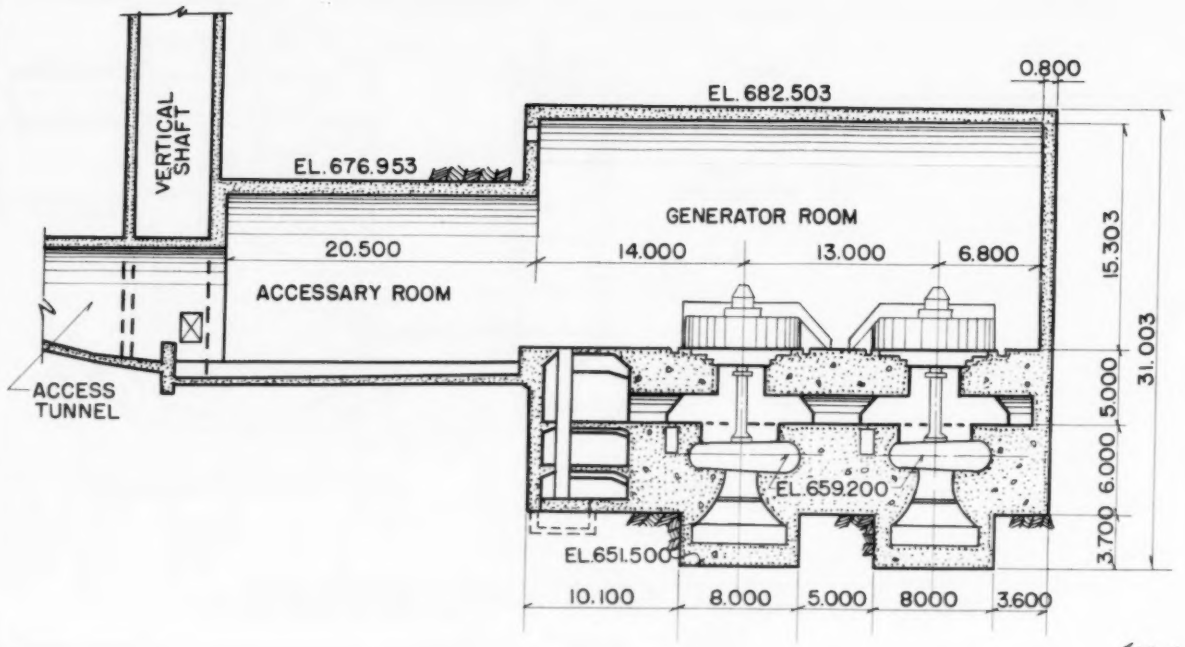
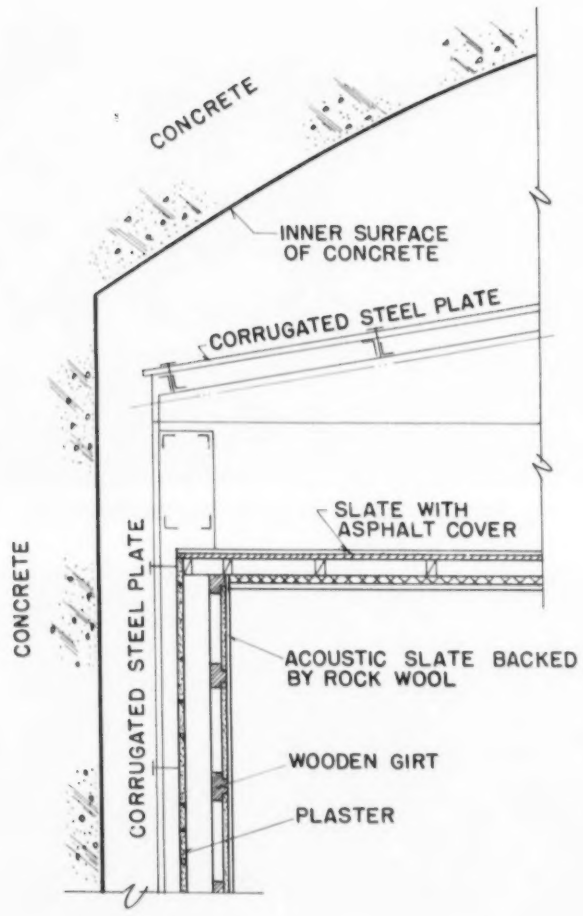


FIG. 4



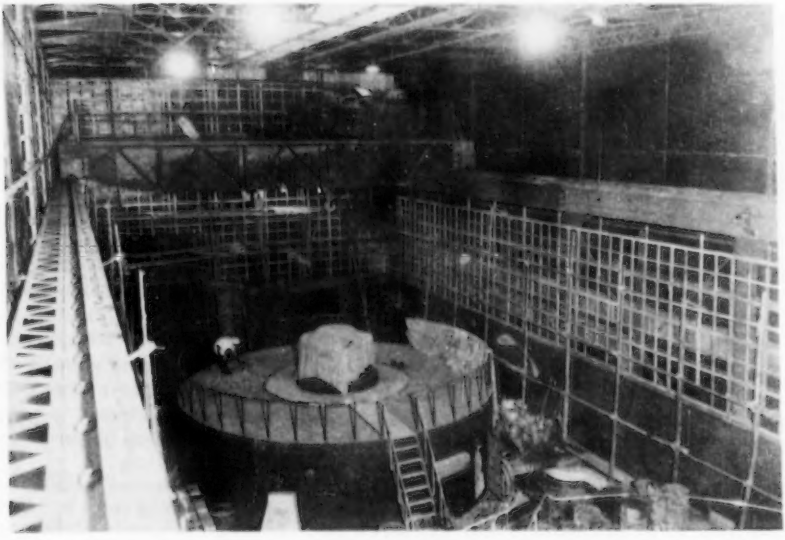
DETAIL OF INNER WALL

FIG. 5

SUDAGAI DAM, POWERHOUSE CONTROL BLDG. & SWYARD. FIG. 6

STEEL SUPPORT FOR POWERHOUSE ROOF FIG. 7

EXCAVATION UNDER CONCRETED ROOF FIG. 8



CONSTRUCTION OF INNER WALLS FIG. 9

Paper 1556 February, 1958 PO 1

Journal of the
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Proceedings of the American Society of Civil Engineers

MONTGOMERY DAM—ROCK FILL WITH ASPHALTIC CONCRETE DECK

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Arthur N. Vanderlip,[1] Members, ASCE
(Proc. Paper 1556)

SYNOPSIS

This paper describes a rock-fill dam with an asphaltic concrete deck representing a new concept in dam engineering. It sets forth the reasons for selecting that type of dam and deck and explains various aspects of design and of construction, especially as regards the asphaltic concrete deck. It cites the advantages and economy attainable with such type of dam construction.

1. Location and Purpose of Montgomery Reservoir

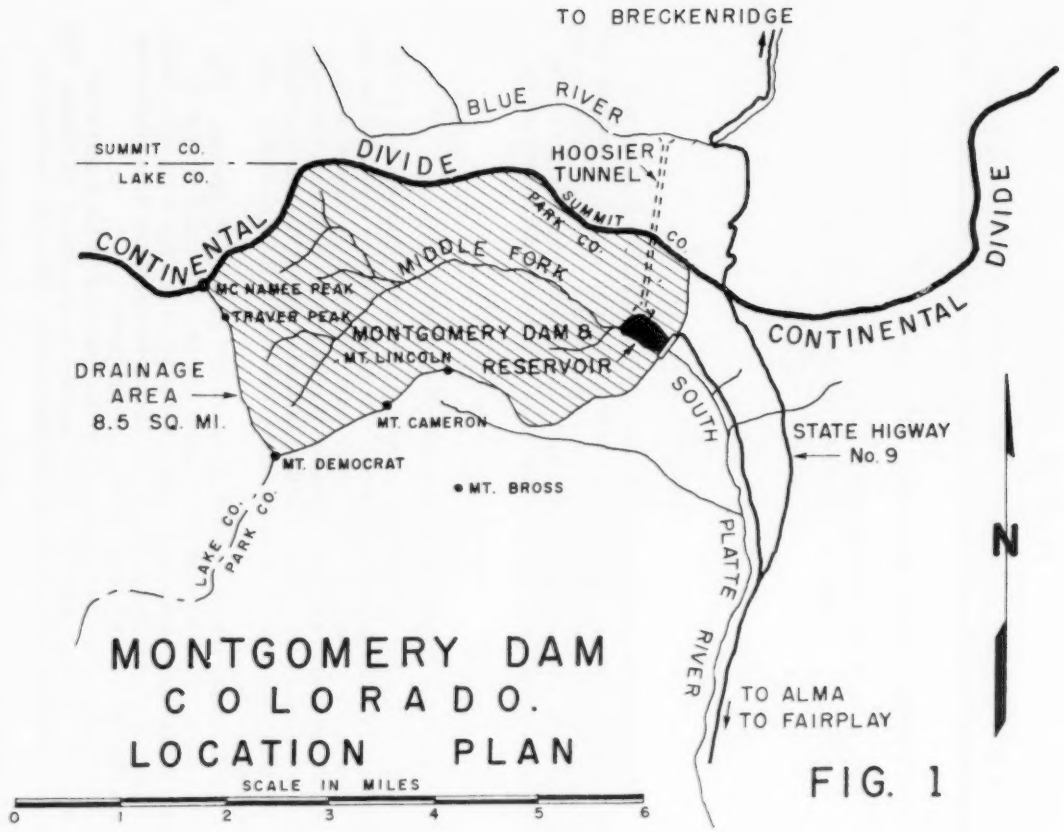
Montgomery dam is located in Colorado, across the upper portion of the Middle Fork of the South Platte River. The site is 5 miles north of Alma and 11 miles north of Fairplay, Colo. It is nearly 100 miles by highway from Colorado Springs and Denver and about 50 miles from the nearest railroad access at Buena Vista, Colo. The dam is located high on the easterly slopes of the Continental Divide; it is only about a mile from Hoosier Pass across the Divide. (Fig. 1)

The dam and reservoir are owned by the City of Colorado Springs. They form a part of the municipal water supply system. The reservoir has the primary function of seasonally storing water which reaches it through a near-by tunnel under the Continental Divide, having been diverted from headwaters of the Blue River, a tributary of the Colorado River.

The reservoir is expected to be filled each summer as the result of the melting of heavy accumulation of snow on the westerly side of the Divide. The water to which the city is entitled will reach the city from the reservoir by

Note: Discussion open until July 1, 1958. A postponement of this closing date can be obtained by writing to the ASCE Manager of Technical Publications. Paper 1556 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. PO 1, February, 1958.

1. Cons. Engs., New York, N. Y.



MONTGOMERY DAM
COLORADO.
LOCATION PLAN

FIG. 1

gravity flow through a 30-in. steel pipeline which feeds into existing reservoirs on the northwest slope of Pike's Peak, thus connecting directly with the city's water supply system.

2. Dam and Reservoir Remarkable in Several Respects

Several exceptional circumstances obtain with regard to the dam and reservoir. Outstanding is the fact that they are located at an altitude more than two miles above sea level. Apparently, in terms of altitude, the structure is the highest dam of substantial size in North America. Incidentally, that altitude has an effect on human performance and otherwise; during construction, the effect was manifest adversely even as regards the efficiency of gasoline engines.

No Middle Fork water may be taken for the use of the city. That is, in the pertinent respect the city's water rights are limited to run-off from the Colorado River drainage area, on the opposite side of the Continental Divide.

The natural drainage area above the dam, being only 8.5 square miles, has no significance except as regards spillway provision at the Montgomery dam. The reservoir is exceptionally small for so long and costly a dam, but such is the price of the much needed water.

Then there is the short annual construction season, of 2-1/2 to 6 months, depending upon the particular operation involved.

Finally, there is the fact that at Montgomery dam there appears to be the first application in the United States of an asphaltic concrete deck to a rock-fill dam. Also it appears to be the first application in the world of this particular type of asphaltic deck to a rock-fill dam.

3. Principal Data and Dimensions (Figs. 2 and 3)

The crest of the dam, exclusive of camber allowance (for settlement) of a maximum of 3 feet, is at elev. 10,868 above Mean Sea Level, U. S. Geological Survey. However, it is contemplated that the height of the dam may in the future be increased, perhaps by as much as 50 feet, and the design, except as to spillway, provides for such raising.

The spillway channel is at the north end of the dam, constructed in part with a lining of reinforced, Portland cement concrete. It has an ungated overflow section of ogee form, with a net width of 97 ft. The crest of the spillway proper, i.e., the ogee, is at elev. 10,861.

The area of the reservoir, when full to spillway crest, is only about 95 acres. The entire reservoir volume, of about 5,100 acre-feet, is utilizable. Should the dam be subsequently raised, as above mentioned, the reservoir capacity would be practically doubled.

The dam is of rock fill, comprised of granite quarried from the mountainside upstream from the right abutment. The watertight diaphragm or deck, resting upon the upstream slope of the dam, is of asphaltic concrete, which was aimed to be constructed in continuous layers and to have the equivalent of a uniform final thickness of 12 inches.

At its maximum section, the dam has a height of 113 ft. above the foundation rock. The length of the rock fill is approximately 1,900 ft. Including the spillway, the length of the dam across the river valley is about 2,000 ft.

The upstream slope of the dam is 1 (vertical) on 1.7 (horizontal); the downstream slope averages 1 on 1.4.

Water from the reservoir is drawn through a screened intake, completely

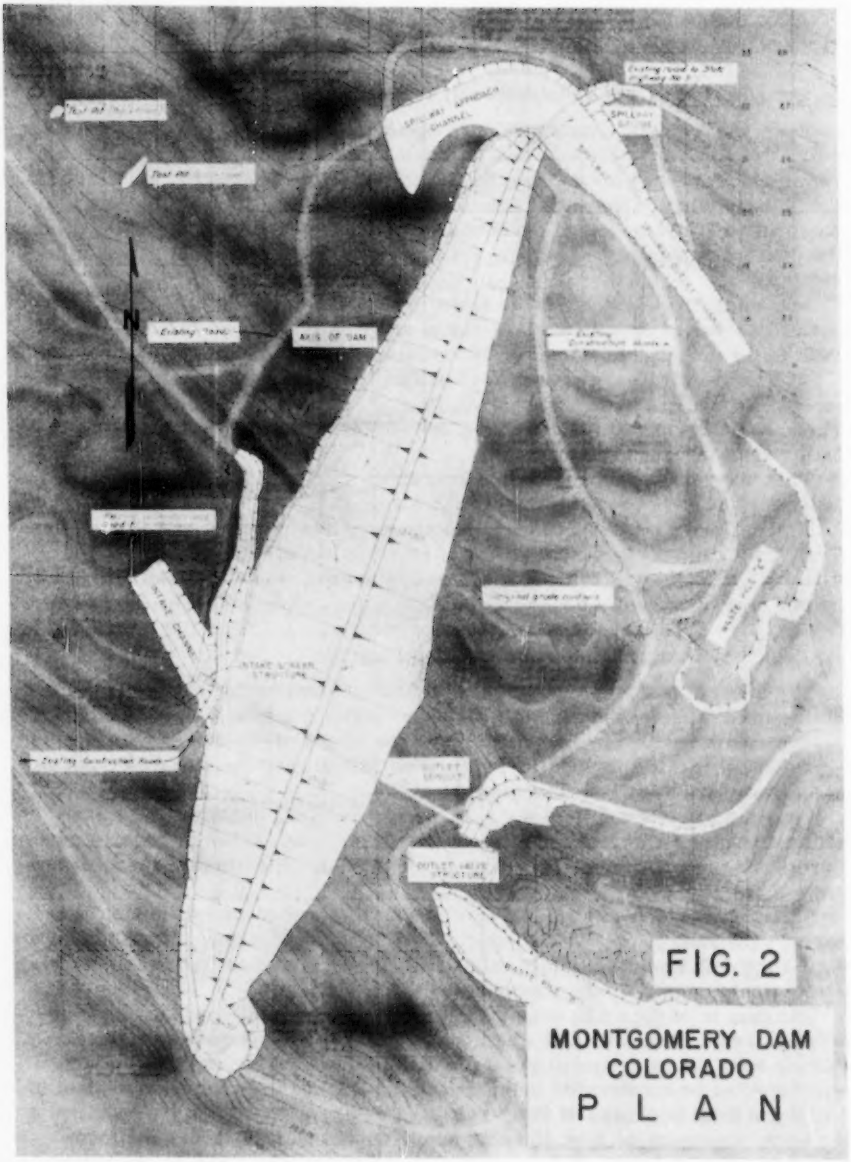

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FIG. 2
MONTGOMERY DAM
COLORADO
P L A N

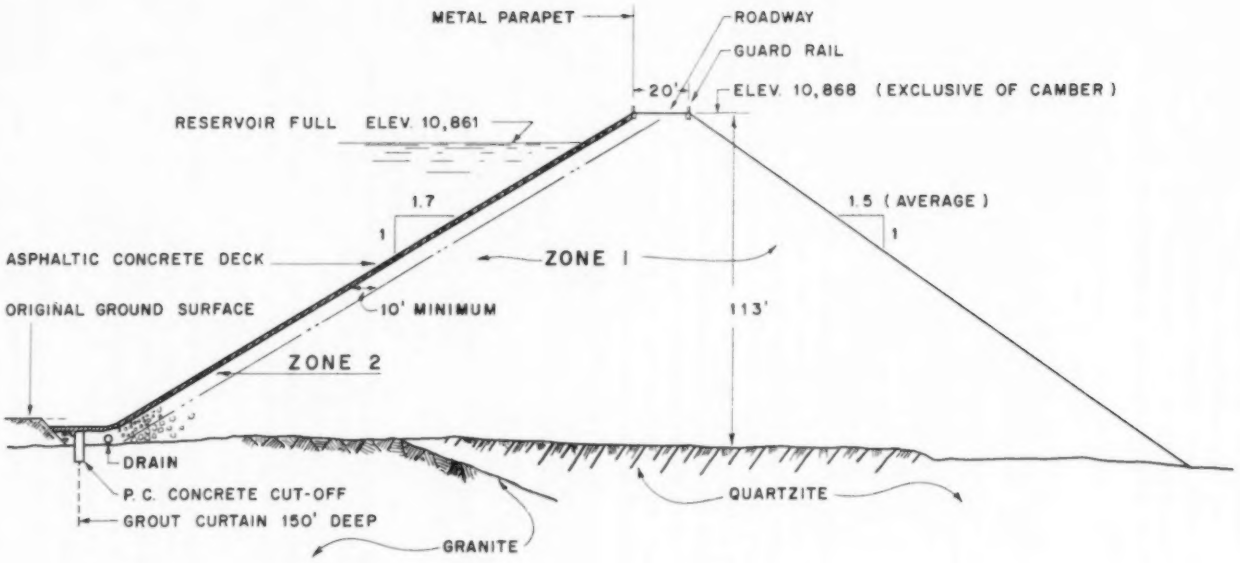


FIG. 3

MONTGOMERY DAM
COLORADO
MAXIMUM CROSS SECTION

submerged except when the reservoir is well-nigh empty. From this intake, a reinforced concrete conduit, of 6 ft. net diameter, 540 ft. long and constructed under the dam in a trench excavated in the foundation rock, conducts the water to an outlet-valve structure. In this structure water can be diverted either into the city's gravity supply line or discharged into the bed of the river. The water which is diverted into the city's pipeline is measured by means of a Parshall flume.

4. Selection of Site

The purpose of the reservoir could have been served by a location almost anywhere downstream from the outlet of the tunnel under the Continental Divide. However, mainly because of the need of head for the gravity-flow pipeline, a location relatively near the tunnel outlet had an important advantage over one any considerable distance downstream.

Serious consideration was given to two sites, one about 3-1/2 miles above the town of Alma and one which would overflow what remained of a ghost mining town called Montgomery. After considerable exploration by drilling, test pits and trenches, together with comparative estimates of cost, the Montgomery site was selected.

The Alma site would have been in sedimentary rock. The Montgomery site is mainly in sedimentary rock, largely quartzite, but roughly one-half of the sealing line at the heel of the dam is in granite, for under the southerly half of the dam the line of contact of the outcrops of the sedimentary rock and the granite lies between the longitudinal axis and the heel.

5. Types of Dam Considered

The width of the valley at the crest of the initial structure being approximately 2,000 ft., an arch dam was out of the question. It was found that any gravity dam of Portland cement concrete, whether of the massive type or of the buttress or so-called hollow type, would be far more costly than a dam of the fill type. Contributing reasons were the long haul for such cement and the short annual construction season, with relatively low temperatures.

Within reasonable proximity to the damsite there is an abundance of glacial deposits, the finer fractions of which might have been barely suitable for rolling into an impervious mass. However, greater estimated cost and the brevity of the season free from freezing temperatures eliminated a dam comprised entirely of rolled earth fill, moistened for the process of rolling. The same factors obtained with regard to the combination of rock fill with an interior, impermeable earth core, whether sloping or central; in addition, the fact that provision had to be made for future raising of the dam in a substantial amount made impracticable the use of any interior earth core except at materially greater initial cost.

Under the circumstances, including the abundance of suitable rock in the abutting mountainsides, it appeared natural and appropriate to select the type wherein the body of the dam is entirely of rock fill and there is an appropriate watertight diaphragm or deck upon the upstream sloping face of the fill. Moreover, with a deck of asphaltic concrete, the estimated cost was less than that of any other conventional type.

In order not to overlook any possibilities, particularly from the standpoint of materials available for the fill, some consideration, though not to the extent of study for design purposes, was given to the use of the unconventional

combination of earth only semi-compacted with a like deck of asphaltic concrete. However, in the light of pertinent conditions for Montgomery dam, such a combination was found not to indicate sufficiently lower cost to offset the advantages of the combination of rock fill with asphaltic concrete deck.

6. Advantages of Rock Fill Dam of Deck Type

The more important advantages of rock fill, specifically of the deck type, were considered to comprise the following:

Provided only that there is a watertight diaphragm or deck on the upstream slope, the stability of a dam of dumped rock fill is assured. Uplift is not a factor.

Further, because of the openness and free-draining nature of rock fill and in the absence of an utterly improbable large break extending through the deck, leakage through the dam would do no damage to the fill proper; it would not endanger the stability of the dam.

Correspondingly, grouting would be necessary only to minimize leakage past the dam, not to eliminate uplift—as in the cases of mass gravity or some hollow dams of Portland cement concrete—or to keep leakage water out of the body fill, as would be essential in the case of an earth-fill dam.

Again, because of the foregoing, rock-fill dams of the deck type require a minimum of site exploration.

The deck type of dam would be particularly suited to permit various construction activities to be carried on simultaneously. Thus the construction of a cut-off wall and extending below it a pressure-grouted diaphragm, each using Portland cement, could be carried on at the same time that the rock fill was being placed, although of course not at the same location within the area of the dam. This advantage was of special importance at Montgomery dam because of the short annual construction season. Furthermore, as concerns the body of the dam, the use of rock fill would permit stretching the all-too-short construction season to its utmost.

In the event of overtopping, whether by floods at part height of the dam during construction or by waves after completion, the rock fill would be less subject to damage, if any, than would be true of an earth fill. This factor also had some bearing upon the extent of provision for freeboard (on account of waves) above maximum flood levels.

As compared with a fill dam having an interior core or a sealing diaphragm, the deck-type of fill would more readily lend itself to future increase in height. In the pertinent respect, increase in height would be accomplished by mere extension of the asphaltic concrete deck upward at the same slope and corresponding extension of the cut-off and sealing provisions up the abutments.

Moreover, the sealing into the foundation being at the heel of the dam, such type would afford future access, if and when needed, to the cut-off wall and the appurtenant above-rock sealing provisions, provided only that the reservoir could be drawn down. It happens that in normal operation the Montgomery reservoir will be drawn down annually.

Finally, the estimated cost of a rock-fill dam of the deck type was well within reason. It was less than the estimated cost for any other type of dam affording equivalent advantages.

7. Selection of Material for Deck

In choosing the material of which the watertight diaphragm or deck of the rock-fill dam was to be constructed, there were considered, as offering possibilities of economic feasibility, three kinds of material, to wit: steel, asphaltic concrete and reinforced Portland cement concrete.

Naturally in the selection of the material in point, estimated cost was a primary factor. It was essentially on the ground of higher cost that steel was eliminated.

Basis of Choice of Asphaltic Concrete

Because of its pertinent characteristics, the use of asphaltic concrete as a material for the construction was approved by Black & Veatch, engineers for the City, when proposed by the authors, as consultants to the former. The authors had had prior experience in the use of asphaltic material in dam construction. Support for the use in the present instance of asphaltic concrete was available also in the history of its use in hydraulic construction elsewhere, as is in part set forth under item 8 hereof. For some instances of use of asphaltic concrete in hydraulic structures, the case history and results over a material extent of time were known. And of course there was available a tremendous amount of information with regard to the use of the material in the construction of highways and airfield runways, though only a portion thereof was directly pertinent.

The selection of the material was made with full realization that existing pertinent information would have to be supplemented by extensive testing. Indeed, the selection was tentative in the sense of being subject to favorable outcome of such testing. It was early recognized that, for the purpose in point, the criteria for satisfactory performance would be different from those currently used for asphaltic concrete in highway work and airfields and probably quite different from those so far used for other hydraulic structures.

During the period November 1953-January 1954, there were carried through tests on locally available aggregates (glacial deposits) sufficient to permit tentative design of a stable mix, mainly for the purposes of cost estimating. These tests established beyond doubt that those local aggregates could be appropriately and economically blended so as to produce excellent gradation and that, when combined with a suitable asphalt, they would yield a workable and stable mixture. Final selection of asphaltic concrete was made only after completion of considerable, though not all, of the pertinent testing. Further reference to that testing appears at item 9 hereof.

Comparison with Portland Cement Concrete Deck

The larger rock-fill dams of the deck type heretofore constructed in this country use an impervious deck of Portland cement concrete on the upstream face. A major element of such structures, involving much time and cost, is the construction of a base for the concrete deck in the form of a thick semi-hand-placed or crane-placed layer of relatively large rocks over the mass of dumped rock. Such layer is considered to be primarily a finish and leveling course, containing fewer voids than the underlying rock mass.

The tendency has been to reduce the thickness of such layer of placed rock. But, even so, the practice is to use a minimum thickness of the order of 8 ft., with greater thickness in lower portions of the higher dams. Also, either

there must be slots for beams to support the joints of the panels of Portland cement concrete or at least, under the planned positions of the joints, there must, by means of like concrete, be provided flat surfaces upon which to construct the joints and to permit them to function properly.

To be sure, the use of an asphaltic concrete deck involves the preparation of a suitable stone base over the upstream face of the rock fill, but such is relatively simple and less costly to construct. There is eliminated the need for the layer of carefully placed base rock and the beams or other joint supports as normally used in a deck of Portland cement concrete.

For Montgomery dam, comparative estimates of cost as between the two kinds of concrete deck indicated a saving of, at the very least, 50% by the use of asphaltic concrete.

At ordinary temperatures, asphaltic concrete is far more flexible than Portland cement concrete. The laboratory investigations made for the purpose of the dam design and likewise field observations show that asphaltic concrete for the intended purpose may be expected to have considerable flexibility at temperatures practically as low as freezing. The evidence indicates that an asphaltic concrete deck will be self-adjusting to any settlement normally to be expected in a reasonably well constructed rock-fill dam. This does not mean, however, that in construction one may disregard the need for keeping settlement, especially localized settlement, to a minimum.

From the evidence available, there is good reason to believe that asphaltic concrete, when properly designed, placed and compacted, will be at least as durable as competitive materials for such deck construction. An interesting example of the use of either an asphaltic mortar (in that case bitumen and loam) or asphaltic concrete (bitumen, sand and gravel) as used for construction in antiquity and surviving to the present time is given by one Andrae in Mitt. Deutsch. Orient. Ges. XXII, p. 33. The material was used on a mile-long embankment along the banks of the Tigris River about B.C. 1300. Andrae states that "After 3300 years, it still faithfully fulfills its purpose". (Asphalts and Allied Substances, by Herbert Abraham, 5th Ed., 1944, Vol. 1, p. 29.)

In the case of Montgomery dam the type of asphalt, the richness of the mix and the thickness of deck are expected to eliminate certain shortcomings which have been observed in the performance of some asphaltic concrete used in road surfaces not subject to substantial kneading action by traffic.

In this connection there is a further consideration, though one not relied upon in the case of Montgomery dam, namely, that there is some reason for expecting that, if some cracking should occur, the asphaltic concrete may be somewhat self-healing. At any rate, assuming in each instance accessibility for repair, a deck of asphaltic concrete is believed to be more easily repaired than is one of Portland cement concrete.

Of great importance, especially in situations like that at the Montgomery site, is the fact that the covering of a given area of deck surface requires far less time with asphaltic concrete than with Portland cement concrete. This may well have saved a year in the total time of construction of Montgomery dam. As it turned out, the summer of 1957 afforded an even shorter period, during which construction was tolerable, than the usual all-too-short season. In terms of conditions suitable for asphaltic concrete work, the total period available was hardly 3 months, whereas normally it would have been about a month longer. Various delays prevented placing of asphaltic concrete until July 22; yet placement of the 27,770 sq. yds. of the asphaltic concrete deck was completed on September 24—or in about two months.

The nature of asphaltic concrete and the manner of placing and compacting it, even with equipment not in all respects designed for the purpose, are such as to permit very rapid, if not continuous, placing. The objective is to place and finish a given strip of asphaltic concrete before the previously placed, adjoining strip has become cold. Also, if necessary, one layer can be placed upon another with only a few hours' lapse of time.

8. Prior Use of Asphaltic Concrete in Hydraulic Structures

The use of asphaltic concrete in hydraulic structures is of course not new. In various forms and for various purposes it has been used in connection with reservoirs. For instance, in the United States, asphaltic concrete has been used to line water-supply reservoirs of the City of Los Angeles, but not as a watertight membrane. Instead, the asphaltic concrete, in a relatively thin layer designed to be porous throughout, was used on earth-embankment slopes in lieu of riprap, i.e., for slope protection only.

Similar use has been made of asphaltic concrete by the Bureau of Reclamation after experimentation at the site of its Bonny dam in eastern Colorado, namely, in lieu of riprap to protect the upstream slope of its Glen Anne dam in California. (See Civil Engineering, June 1952; Western Construction, September 1953.)

There exists in this country only one example where asphaltic concrete is relied upon to provide watertightness for the slopes of a water-supply reservoir, namely, Turner reservoir constructed in 1952 by the City of Salem, Oregon. (See Western Construction, September, 1952.)

Of some pertinency also is the asphalt-mortar cut-off wall constructed about 1940 at the instance of the authors in the left abutment of Claytor dam on New River, Virginia. There a sheathed trench, approximately 3-1/2 ft. wide and 325 ft. long, was excavated, partly through earth but mainly through badly weathered rock, to a maximum depth of nearly 100 ft. below reservoir level. This was then filled with a mixture of asphalt and rock fines in order to prevent leakage from the reservoir around the left end of the dam proper, the latter being of the Portland cement concrete, gravity type. (See Engineering News-Record, March 13, 1941; The Asphalt Institute Quarterly, April 1951.)

More extensive use of asphaltic concrete to provide a watertight diaphragm for rock-fill dams has been made in foreign countries, though it appears that in no case has such use been the same as that at Montgomery dam.

There follows a listing, with short description, of the principal, if not all, rock-fill dams in the construction of which asphaltic concrete has been or may have been used. In all cases, except the last cited, the material was used to form a watertight diaphragm or deck resting upon the upstream face or slope of the dam:

Bou-Hanifia dam, Algeria, 180 ft. high, completed in 1938.—Deck consists of two layers of porous cement concrete separated by 5 in. of asphaltic concrete, in turn in two layers with wire reinforcement between the latter.

Caritaya dam, Chili, 125 ft. high, completed in 1935.—Deck consists of 3/4 in. of asphalt (perhaps asphaltic mortar) between two layers of reinforced concrete.

Diga di Codelago dam, Italy, 64 ft. high, completed in 1893.—Deck consists of a 2 in. mastic layer, covered by stone blocks set in ordinary cement mortar.

Fedaia dam, Italy, 53 ft. high, completed in 1954.—Deck consists of 4 in. of asphaltic concrete between two layers of ordinary cement concrete. Details not available but the outer layer is reinforced, the inner porous.

Genkel dam, Germany, 140 ft. high, completed in 1952.—Deck consists of two thin layers of asphaltic concrete, separated by a layer of gravel (sparsely bound with asphalt) to provide drainage for such leakage as may occur through the outer layer. The outer of the two layers is 3-1/2 in. thick (placed in 3 sub-layers); the inner a little less than 2-1/2 in. (placed in 2 sub-layers). Under the latter and over the upstream face of the rock fill itself, there is a layer of porous cement concrete.

Ghrib dam, Algeria, 235 ft. high, completed in 1936.—Deck consists of two layers of porous cement concrete separated by 5 in. of asphaltic concrete placed in two layers.

Henne dam, Germany, 200 ft. high, completed in 1954.—Deck is similar to that of Genkel dam.

Iril-Emda dam, Algeria, 246 ft. high, completed in 1954.—Deck consists of 5 in. of asphaltic concrete in two layers, between layers of ordinary cement concrete, the upper of the latter being reinforced, the lower porous.

Mulungushi dam, South Africa, 117 ft. high, completed in 1935.—Is stated in an American textbook to have an asphaltic concrete deck 4 to 12 in. thick. However, the references are vague or incorrect and, although research disclosed various aspects of the water development of which Mulungushi dam is a part, no details could be found either as to the asphaltic concrete deck or as to the pertinent construction methods.

Vrla II dam, Yugoslavia, 82 ft. high, completed in 1955.—Deck includes two thin layers of asphaltic concrete (with wire reinforcement between them) covered by a layer of reinforced concrete.

Vale Do Gaio dam, Portugal, 165 ft. high, completed in 1952.—Here asphaltic concrete is used as part of a sloping interior diaphragm or core, consisting of 4 to 8 in. of asphaltic concrete on an inclined cemented masonry wall, with a slope of 1.4 on 1 and buried under an upstream earth fill.

The foregoing of course does not take into account rock-fill dams where asphalt or asphaltic mortar has been applied as a thin coating over steel, wood, Portland cement concrete, etc., where reliance for watertightness was on other materials or where asphalt was used for preservative purposes rather than as a water barrier.

It is appropriate to mention four foreign dams in which asphaltic concrete was used, not on rock fill but, on earth fill, to greater or less extent for the purpose of ensuring watertightness:

Amecker dam, Germany, 40 ft. high, completed in 1934.—Deck consists of 2 in. of asphaltic concrete, but it is not clear that there is reliance thereon for watertightness.

Oued-Sarno dam, Algeria, 92 ft. high, completed in 1952.—Deck consists of 2-3/4 in. of asphaltic concrete.

Schevelinger dam, Germany, 50 ft. high, completed in 1940.—Deck consists of 2-1/2 in. of asphaltic concrete.

Turawa dam, Germany, 43 ft. high, completed in 1937.—Deck consists of 2 in. of asphaltic concrete.

In view of the foregoing, it appears that the use of asphaltic concrete at Montgomery dam, as the sole material for the deck of a rock-fill dam, is the first instance of such use in the Western Hemisphere and that the methods and details of construction at Montgomery dam are unique and involve the first application thereof anywhere.

9. Experimental Basis for Design and Specifications for Asphaltic Concrete Deck

Review of the more pertinent sets of specifications for asphaltic concrete work made clear that the design and the preparation of specifications for an asphaltic concrete deck for Montgomery rock-fill dam would involve considerations and criteria not heretofore fully taken into account.

Among the more important of these may be mentioned:

Certain aspects of watertightness of the asphaltic concrete.

Behavior of the asphaltic concrete when placed upon relatively steep slopes.

Effect of extremely high temperatures upon flow on given slopes.

Effect of variations in types of aggregate, asphalt binder, etc., upon slope-flow characteristics.

Limiting steepness of slopes.

Effect of freezing and thawing and of wave action upon the asphaltic concrete.

Determination of appropriate thickness of the asphaltic concrete deck.

Placing of such thickness, as concerns number of layers, position and treatment of joints, etc.

Special equipment or modifications of existing equipment to be specified.

Nature and preparation of base.

Manner of making watertight connection of the asphaltic concrete with structural features constructed of Portland cement concrete, such as the cut-off wall and the south wing wall of the spillway.

The solution of most of these problems was concluded to warrant laboratory investigation. For some others, there was resort to trial in the field.

The reason why many of the problems related to asphaltic concrete in such a hydraulic structure had not already been adequately covered is undoubtedly to be found in the differences of conditions and requirements to be met in the several fields of application. Most of the previously available information related primarily to the design and construction of highway and airfield pavements—relatively little to the field of hydraulic structures. A recent compilation and discussion in this field appears in the "Proceedings of the First Western Conference on Asphalt and Hydraulics", published as Bulletin No. 78 by the University of Utah, 1955.

On the other hand, basic research as concerns asphalt, aggregates and asphaltic concrete mixtures had been and of course is being done in those two principal fields of earlier activity. Much fundamental experimentation as to the properties and uses of asphalt has been carried on individually and collectively by producers of petroleum, the results of which experimentation are applicable to the design of asphaltic concrete mixes in any specific field.

For the experimental work required for the construction of Montgomery

dam, it was found that nowhere else than at the Engineering Laboratories of the Bureau of Reclamation, Denver, Colorado, was there a laboratory suitably equipped to carry out that experimentation. For this reason and further because of the relative proximity of that laboratory to the site of the dam, arrangements were made with the Bureau for the doing of the work in its Bituminous Laboratory. The arrangement was facilitated, not only because the project in point was that of a municipality of the State of Colorado but also, because the Bureau has before it many like problems involved in the application of asphaltic concrete in the field of hydraulic structures. All of the experimental work in the Bituminous Laboratory was done by laboratory personnel. The Bureau of Reclamation and its personnel were not requested to and did not interpret the test results or directly or indirectly assume any responsibility for interpretation.

The more important categories of laboratory experimentation so undertaken and some of the results were briefly as follows:

a. Tests of materials from various local sources to determine whether suitable aggregates were present therein and could be obtained and processed economically. The test results showed that there could be made available such material in proportions suitable for an excellent asphaltic concrete mix.
b. Tentative design of mix for testing and cost estimating purposes, using such aggregates with 8.5% of asphalt (of steam-refined, paving grade and 50-60 penetration). Later tests were made to establish a like tentative mix but using asphalt of 85-100 penetration.
c. Tests of such properties as ductility, softening point, penetration and flashpoint; thin-film and spot tests of various asphalts; also determinations of values of the asphaltic concrete as concerns density, stability (compressive strength), flow, immersion compression and sustained load or creep tests, on a sufficient number of mixes (with various percentages of asphalt) to permit establishing the optimum mix. The tests and procedures involved were largely those which have been standardized.
d. Extensive tests involving a number of types (including steam or vacuum refined and catalytically air blown) and penetration grades of asphalts, some with additives (including natural and synthetic rubber), all for the purpose of determining the most likely candidates for ultimate use.
e. Slope-flow tests to determine the behavior and characteristics of selected asphaltic concrete mixes under the conditions of various slopes and temperatures. These tests were made in three phases and consumed a great deal of time but were eminently worthwhile.
f. Various special or non-standard tests, to determine other important properties or limitations. The more important of these were tests of ice action; wave action; alternate freezing and thawing, on 4-in. diameter cylindrical specimens; transmission of 'cold', on one of the rectangular slope-flow specimens; contraction due to low temperatures; permeability; flexure, on small rectangular beams at minus 110 deg. F.; and flexibility, on 3-in. thick discs, 18 in. in diameter, at various pressures and temperatures.

The above and other tests led to the firm conclusion that asphaltic concrete could be placed on slopes as steep as 1 on 1.5 and would remain stable thereon even if the temperature of the surface of the material were somewhat in

excess of 140 deg. F. However, it was determined that for Montgomery dam the deck slope should be 1 on 1.7. Further it was concluded that, with the materials available at the Montgomery site, best results would be obtained by means of asphaltic concrete containing approximately 8.5% (in terms of weight of dried aggregate) of steam-refined, paving-grade asphalt as binder.

A more detailed description of these tests, together with the test results and interpretations, will be presented in February, 1958, by the authors before the annual meeting at Montreal, Canada, of the Association of Asphalt Paving Technologists, a technical group specializing in asphalt and asphaltic concrete.

The 'job mix' finally selected was essentially the same as that used in the experimental slope-flow tests. The asphaltic concrete aggregates, as processed from a nearby glacial deposit, were limited to a maximum size of 1-1/2 in. The gradation approximated the following:

Size	Per Cent Passing Square-Mesh Screen (Washed Sample)	Size	Per Cent Passing Square-Mesh Screen (Washed Sample)
1-1/2 in.	100	No. 10	47.2
3/4 in.	87.5	No. 40	30.8
1/2 in.	75.9	No. 80	20.6
No. 4	59.2	No. 200	11.8

In order to determine the effect of thermal changes upon a more extensive surface than was available in the laboratory, a 20 x 120 ft. asphaltic concrete test panel, including horizontal as well as vertical joints, was constructed in late October 1955. It was placed upon a well-drained, slightly sloping area not far from the site. This panel consisted of two strips of asphaltic concrete, each of three layers totaling 7 to 8 in. in thickness after compaction. Over a period comprising two winters, careful observations and precise measurements, between pairs of brass plugs set flush with the top surface, disclosed no detrimental surface changes, variations in length or joint movement.

10. Certain Phases of Design

Slopes

In general, dumped rock for a fill assumes naturally a slope varying between 1 on 1.3 and 1 on 1.4, as was later found to be true in the case of the Montgomery fill. It followed that the slopes of both downstream and upstream faces of the dam should not be steeper; otherwise expensive placement of the rock for such slopes by derrick would become necessary, in order that the slopes would be stable.

For the downstream face of the rock fill, an average slope approximately 1 on 1.4, somewhat flatter than the anticipated natural slope, was used for the entire face.

Quite different were the considerations for determination of the slope of the upstream face of the dam, for that slope involves the asphaltic concrete deck. Of course the steeper that slope, the less would be the quantities involved in the construction. For proper placing of the asphaltic concrete, such slope needed to be uniform, i.e., without berms. Moreover, it needed to be such that the asphaltic concrete thereon would remain stable and not flow

downslope even under the highest ambient temperatures. Also the slope needed to be sufficiently flat to permit working on the partially finished asphaltic concrete deck without requiring such facilities as walkways, ladders, etc. The tests referred to at item 9 above had shown that asphaltic concrete would remain stable on a slope as steep as 1 on 1.5. Safety of movement of persons on the slope and facility of machine placement of the asphaltic concrete were believed to be attainable with a slope of 1 on 1.7.

After taking all of the foregoing into account, the latter slope was selected for the upstream face of the dam. As to the safety aspect, construction at that slope proved to be entirely feasible. However, ropes anchored at the crest of the dam were used by contractor's employees to permit greater speed and less effort in moving about on that slope.

Camber

All rock-fill dams settle during and for a considerable time subsequent to construction. Usually, to provide enough height of fill to offset future settlement, an over-fill is prescribed. Without this, settlement might result, not merely in reduction of freeboard but also, in an unsightly sagging appearance, especially at or near the section of maximum height. For Montgomery dam, such over-fill was 3 ft. at the maximum section of the dam, reducing uniformly to zero at each abutment. (See item 12 hereof.)

As a result of this camber, the upstream slope of the dam, as initially constructed, became slightly steeper than 1 on 1.7 at the section of maximum height, viz., 1 on 1.66, flattening gradually to 1 on 1.7 at the abutments. The downstream average slope, intended ultimate to be 1 on 1.4, was thus actually constructed at 1 on 1.36 at the maximum section of the dam.

Cut-off Wall and Its Connection to Asphaltic Concrete Deck

The cut-off wall extending into the foundation rock was located, not at the intersection of the upstream slope of the dam with the foundation rock but instead, a short distance upstream from that intersection, so that the several parts of that wall would be straight in plan. Thus, the horizontal projection of the downstream face of the cut-off wall is a polygonal envelope of the highly irregular line of intersection of the upstream slope of the rock fill with the surface of the foundation rock.

The wall itself is a simple Portland cement concrete structure, reinforced, rectangular in cross-section, 3 ft. wide, extending 3 to 8 ft. into rock, depending on conditions after blasting, and projecting a few feet above the foundation rock, the precise amount depending on the irregularities of the surface of that rock. Although the slope of the top of this cut-off changes from place to place, no steps occur from one part to the other, i.e., there is a continuous surface from abutment to abutment.

For connection of the asphaltic concrete deck to the top of the cut-off, the design was such that the top of the fill, i.e., the base for the asphaltic concrete, would have a transition from its prescribed slope (of 1 on 1.7) to horizontality, flush with the top of the cut-off wall. (Fig. 4)

The asphaltic concrete, curving with the surface of its rock-fill base, was placed upon and over the top of the concrete cut-off wall and extended horizontally a few feet upstream therefrom. The theory of this method of construction is that, by reason of the open nature of that rock fill which is just downstream from the cut-off wall and under the asphaltic concrete, the water

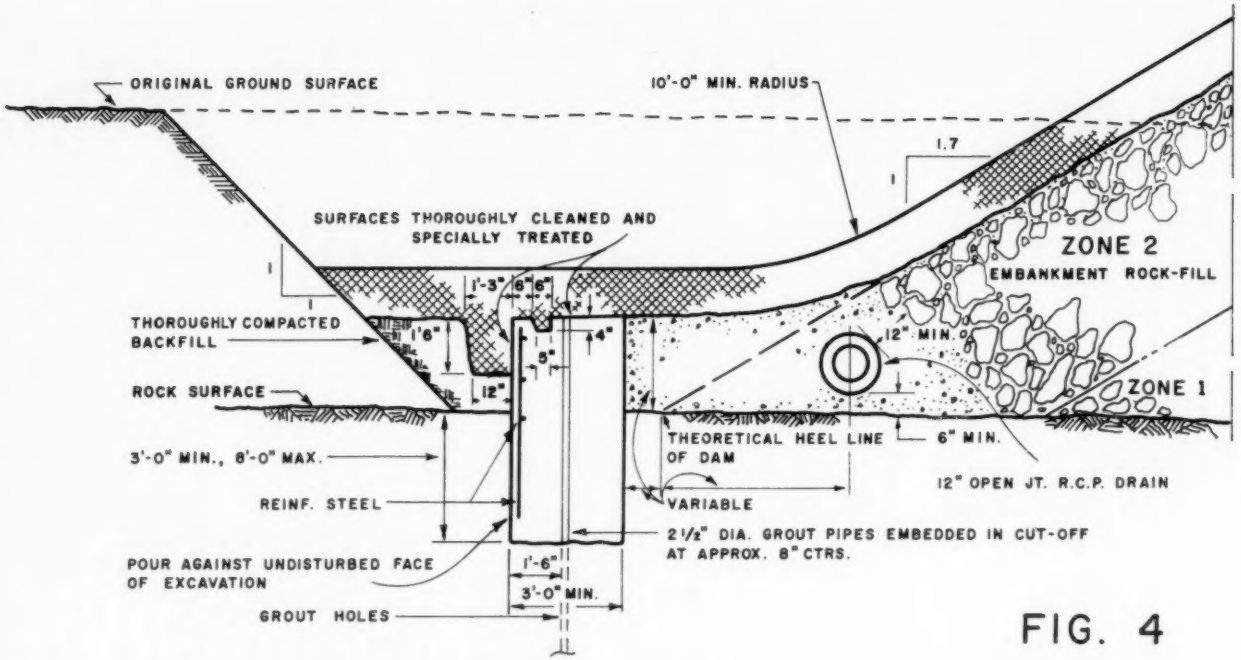


FIG. 4

MONTGOMERY DAM COLORADO
DETAIL AT CUT-OFF

pressure on, together with the dead weight of, the asphaltic concrete and proper bonding will result in a watertight connection between cut-off wall and asphaltic concrete deck.

As an additional precaution, an asphaltic concrete "hook", approximately 12 x 18 in. in cross section, was constructed just upstream from the cut-off wall, so as to extend from the deck asphaltic concrete down into compacted backfill. This increases the area of asphaltic concrete bonded to the Portland cement concrete cut-off wall and thereby further diminishes the chances of leakage between deck and cut-off wall.

Drainage

Although the dumping of loose rock, the usual construction method for such rock fill, is ordinarily assumed to result in free drainage, there are possibilities that such may not everywhere be the case and that there may exist circumstances under which ensurance as to free drainage is of primary importance.

The sluicing water, used in placement of the rock fill, drains down to the foundation of the dam and thence flows off; thus it is possible that the finer material moved by the sluicing water may fill the voids of some lower portions of the rock fill. Although such filling of the lower voids can do no harm as to stability or settlement of the fill, it tends to create within the rock fill an interior barrier which extends to uncertain height and could become impervious.

Consequently, if the reservoir formed by the rock-fill dam is to be emptied from time to time, there exists the possibility that water from leakage or seepage, whether through the deck or in the form of groundwater from the foundation, will pond within the rock-fill voids between the deck and such interior barrier and thus, when the reservoir is drawn below the level of such pond, will exert pressure against the lower side of the deck.

The Montgomery reservoir is intended to be drawn down once a year. Therefore, to eliminate the possibility of such back pressure against the asphaltic concrete deck. there has been provided, below this deck, drainage which makes reasonably certain that such back pressure can not occur. The provision in point consists partly of prescriptions as to the rock to be placed immediately under the deck and partly of a positive drainage system.

The rock-fill volume was divided into two zones. For Zone 1, by far the larger part of the fill, the material was permitted to contain up to 10% of rock fines (4 in. and smaller). For Zone 2, lying between Zone 1 and the asphaltic concrete deck and having a horizontal width of not less than 10 ft., the material was prescribed to be completely free of any and all rocks of 3-in. and smaller size.

The positive drainage consists of an open joint, concrete, drain pipe of 18 in. diameter, installed at the bottom of Zone 2 along the entire cut-off. Near the lowest point of that longitudinal drain, a like connecting, transverse pipe, of 24-in. diameter, provides a drainage outlet into the river bed downstream.

Spillway

For so small a drainage area (8.5 sq. mi.), at so great an altitude (10,861 ft. above M.S.L.), determination of spillway provision is not subject to the

normally controlling considerations.

Although rains of day-long duration are known to have occurred, day-long hard rains in that region are unknown. The gated spillway of the nearby Green Mountain dam of the Bureau of Reclamation, located on the Blue River on the westerly side of the Continental Divide, is understood to provide for a peak discharge of 25,000 cfs. from a drainage area of 583 sq. mi. On the assumption that flood peak discharge varies with the square root of the drainage area, the comparable flood peak for the drainage area at Montgomery dam would be approximately 3,000 cfs.

Usually, for ungated spillways, the rise in reservoir level accompanying flood discharges reduces substantially the peak discharge over the spillway as compared with the peak flow into the reservoir, but no substantial reduction in this respect will occur at the Montgomery reservoir, for the reservoir area at spillway crest level is less than 100 acres. A discharge of 3,000 cfs. passing over the spillway would cause the reservoir level to rise 4 ft. to elev. 10,865, or 3 ft. below the crest of the rock fill (exclusive of residual camber). However, there is a parapet, 2 ft. 2 in. high along the upstream edge of the crest, constructed of welded, corrugated steel plates and held vertical by means of steel posts anchored firmly in precast Portland cement concrete blocks which in turn are embedded in the rock fill. The parapet has its bottom edge embedded in the top of the asphaltic concrete deck of the dam. Thus, with such discharge, freeboard and hence margin of at least 5 ft. are provided above that flood level.

11. Construction (Except Deck)

Construction of Montgomery Dam was begun in August, 1854. In its entirety it required a little over three years, not due to requirements and difficulties of the job but, because the annual construction season is so very short.

The rock fill proper was begun at the start of the 1955 construction season and completed at the end of the 1956 season. The asphaltic concrete deck, together with minor other work, was constructed during 1957. Of the total calendar period, hardly one-half involved actual construction.

Quarrying and Processing of Rock for Fill

The contract specifications permitted use in the fill of rock up to 5 tons (Fig. 5). On the other hand, they limited the amount of rock fines (4 in. and smaller) to be used in the fill (see under item 10, Drainage). It was found that upon blasting the granite there resulted fines exceeding the 10% limitation. Although selection of material in loading of trucks by the quarry shoven made it possible to bring the loaded rock within the specification, the residual material was generally useless without some form of screening.

During the early stage of construction, the contractor had installed a trommel or rotating screen, largely because of needs for rock of the smaller sizes, notably to embed open-joint drain pipes (see item 10 hereof) and for preparation of the upstream face of the rock fill as a base for the asphaltic concrete deck (see item 13), but also because the material for Zone 2 was required to be completely without fines. However, the use of the trommel to provide, from the residual quarry material, usable rock for Zone 1 of the fill proved entirely too slow and too expensive.

Next a 'grizzly' or bar screen, of steel rails, was tried and found wanting.

It was not until, in 1956, a so-called wobbler screen had been installed that the job could proceed at the required rate and with assurance that the fill material would meet specifications, even if this involved some double handling of rock. For description of the wobbler screen and its operation, see Engineering News-Record, November 15, 1956, p. 44, and Western Construction, December, 1956, p. 48.

Rock Fill

The rock fill was placed in lifts which were prescribed not to exceed 40 ft. in height. Actually the maximum approximated 30 ft. The contractor was left practically free to work out his own plans in this respect.

Essentially all of the rock was placed by dumping. During and immediately after the dumping of each truck load, the rock was liberally sluiced with water. Each nozzle stream was prescribed to be capable of delivering 500 gpm. at a pressure (measured at nozzle base) of at least 60 pounds per sq. in. (psi.). The sluicing quantity was prescribed to be at least twice that of the gross volume of the rock placed and so sluiced.

Largely the contractor was left free in the choice of method to construct the upstream slope of the dam (1 on 1.7), being flatter than the natural slope of dumped rock. Generally this was accomplished by a temporary overfill at the natural slope, whereupon such overfill at the top of the lift was moved down slope to make up for underfilling at the bottom of the lift.

Grouting

The grouting through and along the cut-off wall at Montgomery dam, although a part of the general contract, was sublet to a company specializing in that type of work.

The grout curtain extends vertically 150 ft., below the base of the cut-off wall, across the deeper part of the valley, decreasing to 100 ft. at the abutments. Grouting was prescribed to take place at the maximum pressure attainable, not in excess of 200 psi. Whereas this prescription was somewhat severe for a dam not over 113 ft. high, it took into account the fact that the grouting would have to serve as well in case of future raising of the dam by some 50 ft.

The common procedure was followed in that, first, the upper 50 ft. of the curtain were grouted by means of primary holes plus intermediate or secondary holes where necessary. In general, in the upper 10 ft. of rock grouting pressures of 75 psi. were attained without uplift of cut-off wall. Some leakage of grout to the surface occurred, but, if any grout entered nearby rock fill, it was not observed.

Thereupon grouting at lower stages of the grout curtain took place. This was discontinued near the end of the 1956 construction season, although in places there was indicated additional work which would have improved the curtain. Termination was deemed by the City to be appropriate in view of the limited purpose of grouting at a deck-type, rock-fill dam (see item 6).

12. Settlement of Fill

All rock-fill dams will settle during construction and further still upon completion. Presumably the reason is that, due to increasing load, the

individual pieces of rock bearing upon each other will become blunted at their contact points. Measurements made at rock-fill dams show that this settlement process continues at a diminishing rate over several years.

Settlement is affected by various aspects of construction but, assuming reasonable uniformity of construction, settlement may be expected to vary about directly with height of fill.

Besides the essentially vertical settlement due to dead weight, there may be expected additional settlement, vertically and as well horizontally in a downstream direction, upon filling of the reservoir and the loading of the deck and hence the dam as a whole. As is customary, the design of Montgomery dam provided a camber, in this case 3 ft. maximum (see item 10).

The Montgomery reservoir has not yet been filled; hence settlement data to date are no more than preliminary, being due only to the gravity effect on the fill. At the section of maximum height of completed fill, settlement measured during June 1957 (i.e., at the beginning of the year's construction season), as compared with the elevation of the same point in October 1956, was only 0.045 ft. or less than 1 in. At no location along the entire dam did the settlement over that period exceed 0.055 ft.

13. Preparation of Base for Asphaltic Concrete Deck

It was recognized that the surface of the sloping upstream face of the rock fill, upon which the asphaltic concrete deck was to be placed, must among other things be sufficiently stable to support the paving equipment during the placing of the first layer of asphaltic concrete.

The asphaltic concrete deck was prescribed to rest essentially on Zone 2 material (minimum horizontal thickness, 10 ft.) but it was specified that over that material there should be spread 3/4 - 3 in. rock, placed in layers and compacted in such a manner that the larger interstices at the surface of the Zone 2 material would be filled and the surface consolidated (Fig. 6). The objective was to avoid local settlement during subsequent operations of placing and rolling the asphaltic concrete. A further objective was a fairly smooth surface, free from any substantial amount of undulation or unevenness.

Near the end of the 1955 construction season, during which a portion of the top surfacing material was placed, serious doubt arose as to whether subsequent compaction alone would result in a stable surface. It was concluded that a 'penetration' coat of an appropriate asphalt binder might be required to be applied to the so-consolidated base just prior to the placing of the asphaltic concrete deck.

No surface consolidation of the base was attempted until near the end of the 1956 working season, when the rock fill was nearing completion and the major portion of the 3/4 - 3 in. surfacing material had been placed. After some experimenting, it was thought that consolidation by several upslope passes of a vibratory roller probably would afford adequate compaction (Fig. 7). However, due to the fact that winter closed in during September of that year, some of the intended work was deferred, to be done during the next construction season. It was realized that in all probability the effects of winter would require some refinishing, at least of parts of the surface, prior to placing the asphaltic concrete deck during the summer of 1957.

Accordingly, it was arranged that the contractor for the asphaltic concrete deck should do any repair and supplementary work upon the base surface, accomplish further compacting of that surface, apply a penetration coat of

Fig. 5 - Typical rock fill.

Fig. 6 - Partially completed base for asphaltic concrete deck. Sluicing monitors barely visible at foot of advancing face of rock fill.

asphalt and do whatever else might be necessary to complete the base suitably to receive the asphaltic concrete deck. Such further work is described at item 15 hereof.

Fig. 7 - Vibratory roller compacting surfacing material.

14. Specifications for Asphaltic Concrete Deck

As indicated under item 9, the specifications for the asphaltic concrete deck of Montgomery dam reflected the results of experimentation. Also they reflected the influence of pertinent parts of what were considered the best of the latest specifications for asphaltic concrete pavements of turnpikes and other important highways. However, in some important respects, even the requirements of the latter were materially strengthened for use at Montgomery dam.

It is believed that in no respect are the pertinent specifications for Montgomery dam less rigid and in many they are more rigid than any other specifications so far written for the design and placing of a hot bituminous mixture. For instance, asphalt technologists have been somewhat concerned because of changes known to take place in certain of the physical properties of asphalt when thin films thereof are subjected to relatively high temperatures. This condition is approached when the asphalt is introduced (in some instances, by spraying under pressure) into and mixed with a mass of hot aggregate. The result is an ageing action, normally evidenced by loss of weight, increase in softening point and decreases in ductility and penetration. Most specifications for asphalt itself had not embodied (for acceptability) any limitations upon such changes.

However, the Montgomery specifications, by prescribing limits upon the permissible changes in properties of the residues as the result of thin-film tests (oven heating at 325 deg. F. for 5 hours), took cognizance of the foregoing

facts. The specifications were intended to confine the selection of asphalts to those exhibiting small, yet reasonable, changes in those critical properties.

Despite the disadvantage of appearing to be unduly complicated, there were included in the specifications for asphaltic concrete a number of other largely new requirements aimed to secure, under the particular nature of the work, results of the highest quality consistent with reasonable cost. Such included the prescribing of -

Automatic recording, at a single central location, of all important temperatures involved in the operation of the mixing plant.

Unusually close control of conditions of placing, including temperatures and moisture—this because, even in summer, the temperatures are in a relatively low range.

A high degree of density of the asphaltic concrete in place—despite such difficulties of placement as result from the relatively steep slope.

Pertinent in this connection, although not required by the specifications, is the fact that, in order to limit the extent of fractured faces, all boulders larger than 6 in., as found in the glacial deposit, were wasted. The processed material was supplied to the cold-feed bins in 3 sizes, to wit, 1-1/2 to 5/8 in., 5/8 in. to No. 4 and minus No. 4.

The specifications provided that, where needed, there be applied between asphaltic concrete surfaces a 'tack' coat of paving-grade asphalt or rapid-curing, liquid asphalt, in either case to be of standard grade as established by The Asphalt Institute. The rate of application was prescribed not to exceed 0.1 gal. per sq. yd.

For the drafting of the specifications and in order to obtain the benefit of the most advanced thinking in the design and construction of such asphaltic concrete, there were sought suggestions and comments from various qualified persons, specialists in this field. In this respect the authors acknowledge especial indebtedness to, among others, Messrs. John M. Griffith, A. S. Wellborn, the late J. R. Banning and Donald D. Dagler of The Asphalt Institute, the headquarters of which are at College Park, Maryland; Jewell R. Benson, Consulting Engineer, Denver, Colo.; L. M. Ellsperman of the Bituminous Laboratory, Bureau of Reclamation, Denver, Colo.; J. O. Izatt, of the Shell Oil Company, New York; and R. A. Andreasen and R. F. Jouannet, of the Barber-Green Co., Aurora, Illinois.

15. Construction of Asphaltic Concrete Deck

Bids on the construction of the asphaltic concrete deck having been received in November 1956, the contract was awarded before the end of that year. Thus during the winter the successful bidder had opportunity to study the requirements of the job and to plan and provide appropriate equipment.

Except for modifications to paving equipment and trucks and in the methods of handling the same, in order to secure efficient operation on the sloping upstream face, paving methods and techniques were those normally required for high-grade, hot-mix, asphaltic paving of highways and aviation runways. The contractor already owned an appropriate mixing plant and appurtenant facilities, so that it was necessary only to supplement the same in certain respects, including provision for thermal control and other special equipment required under the specifications.

However, the need for modifying existing highway paving equipment and

providing some entirely new forms of equipment was more acute as concerns provision for operating heavy equipment on the upstream slope (1 on 1.7) of the dam, i.e., in the placing and compacting of the asphaltic concrete deck. In this connection the contractor designed and constructed a novel turntable, to be located upon and moved, by caterpillar tractors, along the 20-ft. wide roadway on the crest of the rock fill. This was designed to transfer the paving machine from one lane (up and down slope) to another and, by permitting the requisite change in direction, to transfer trucks (carrying asphaltic concrete) from the roadway on the crest to and from the sloping deck. The turntable was supplied with electric powered, variable speed drums and winches so that, by movement of the loaded and empty trucks down and up slope independently of the movement of the paving machine, there was permitted largely uninterrupted paving from bottom to top of the slope (Fig. 8).

The specifications permitted much latitude as to type of roller to be used in order to attain the required density of the asphaltic concrete. After some experimentation, the contractor was successful in adapting a standard type of vibratory roller to the needs of the case. The movement of the roller up and down slope was powered and controlled by a caterpillar tractor operating on the crest roadway and equipped with a boom and cables, leading from its power take-off to the roller.

As mentioned at item 13, it was arranged that the contractor for the asphaltic concrete deck should do certain work which had been deferred from the close of the previous (1956) construction season. The most important of the items involved in this respect was that of completing the attainment of a surface or base capable of supporting and remaining stable under the weight and operating movement of the paving equipment. The effect of movement of the 11-ton paving machine was of course greatly aggravated by reason of the slope.

The situation as regards completion of such preparation of the base became the more acute because of the unexpected delay in the melting of the winter's heavy accumulation of snow and in the arrival of temperatures permitting the work to proceed.

It had previously been concluded that the surface of the rock base would need to be treated with an appropriate penetration type of asphalt. After an additional pass of the vibratory roller over the entire surface, such a penetration coat was applied.

Even after such application, however, the surface still showed insufficient stability. After consideration of several expedients which seemed feasible, it was concluded to place upon the base what was referred to as a 'leveling course' of asphaltic concrete, of the same mix as specified for the deck proper. This was intended to be relatively thin. Because the sloping base would not yet tolerate the movement of the paving machine without disturbance of the surface, resort was had to placing of the leveling course by means of trucks.

For this operation, trucks loaded with the hot asphaltic concrete were backed down the slope, by means of the cable and winch controls located at the turntable. During such lowering along the slope, the partly open tailgate of the trucks permitted spreading of the material ahead of the backing truck. This spreading had to be supplemented by hand work. Subsequently the so-spread material was subjected to a single pass of the vibratory roller.

The leveling course varied in thickness from less than 1 in. to about 3 in. It resulted in a base upon which the paving machine and supply trucks could

Fig. 8 - Some of the equipment involved in the construction of asphaltic concrete deck.

and did operate to place the first regular and of course subsequent layers of asphaltic concrete deck.

Subject only to adequacy of compaction "of all parts of each layer", the specifications permitted the placing of the deck proper in two layers. The minimum permissible thickness was 3 in. (after compaction). However, the contractor, wisely it is thought, chose to place the deck proper in three layers.

In this connection, there arose the question as to what value should be attributed to the leveling course in terms of an equivalent portion of thickness of regularly placed and compacted deck. Except for special conditions obtaining near the north end of the deck, it was concluded in behalf of the City in effect that it would be appropriate to allow 1-1/2 in. as much equivalence of the leveling course. The result was a total machine-placed, asphaltic concrete thickness approximating 10-1/2 in. This was placed in layers having thicknesses approximating successively 4 in., 3-1/2 in. and 3 in., all measured after compaction.

The asphaltic concrete was placed on the upslope pass of the paving machine in 12-foot widths or strips. All joints within the layers were required to be offset to the minimum extent of 3 ft. in relation to the joints in lower layers. Generally, just prior to placing the adjacent asphaltic concrete, cold joints between or within strips were given a thin coat of hot asphalt of the type used in the concrete itself.

Despite the unfortunate lateness in opening of the 1957 construction season and therefore its brevity and despite the complication in attainment of a stable base for the regularly placed asphaltic concrete of the deck, the completion of the deck was accomplished by the time weather conditions were becoming intolerable.

For over-all views of the completed structure, see Figs. 9 and 10.

16. Cost Data

The more important bid comparison figures of the successful bidders are set forth below:

Rock fill
- Zone 1 material, 420,000 cu. yds. @ $1.20
- Zone 2 material, 53,000 cu. yds. @ $ 3.00
- (Average for entire rock fill - $1.40 per cu. yd.)

Grouting
- Drilling with AX bit, 3,000 ft. @ $5.85
- Drilling with BX bit, 26,000 ft. @ $7.15
- Grouting, 7,500 sacks of cement @ $6.00

Preparation of rock base for asphaltic concrete deck
- By dam contractor, 27,000 sq. yds. @ $1.60
- By deck contractor, supplemental, 2,000 tons of rock of prescribed sizes $10.00
- Asphalt penetration coat, 20,000 gals. @ $0.35
- (Average, for entire deck area - $2.60 per sq. yd.)

Asphaltic concrete deck
- Asphalt, alone, 1,400 tons @ $43.00
- Asphaltic concrete, in place, except asphalt, 19,000 tons @ $11.00
- Vibratory rolling, 60,000 sq. yds. @ $0.25
- (Average, for entire deck - $10.50 per sq. yd.)

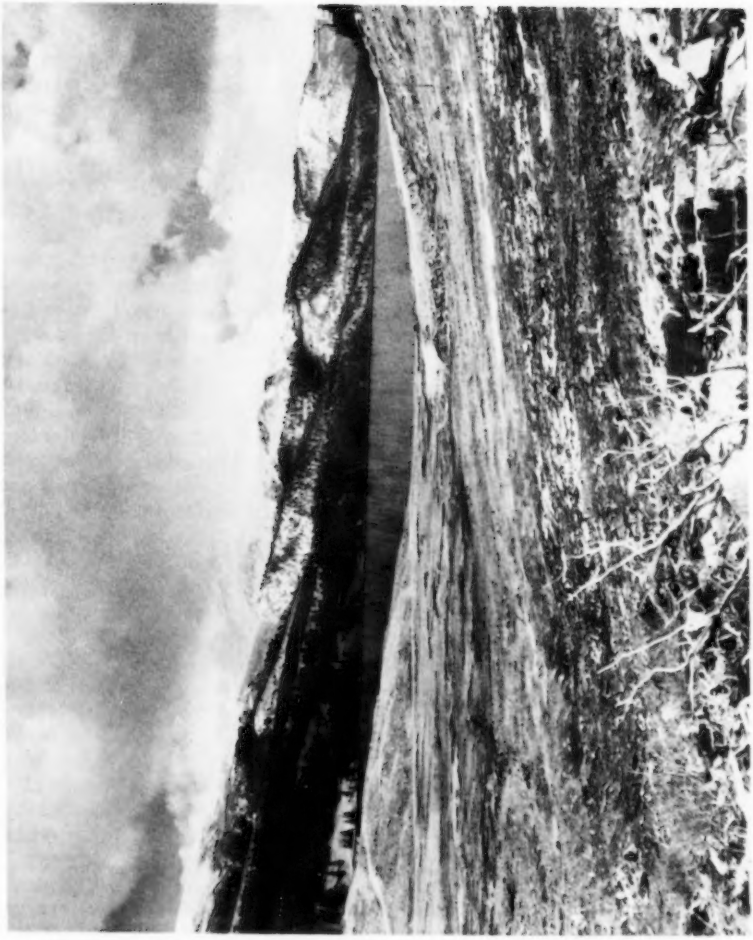
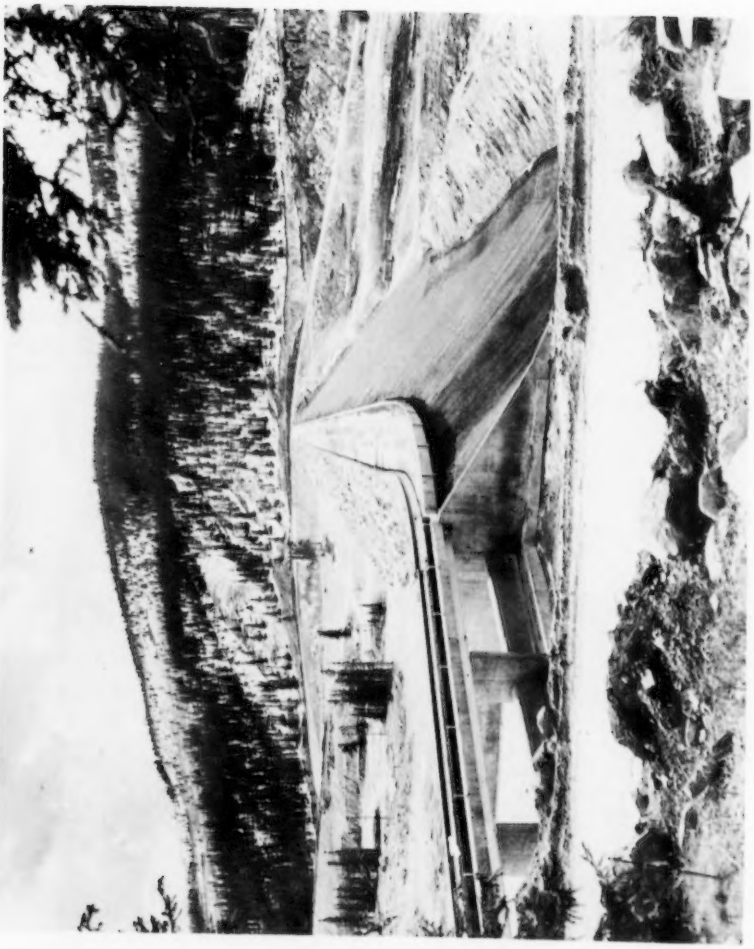


Fig. 9 - Upstream view of dam.



The total construction cost of the dam, including exploration, was $2,663,910.

17. Personnel Involved

Throughout the development of the project of which Montgomery dam is a part, the city's pertinent activities have been receiving the general direction of J. B. Biery, City Manager. During the entire period, the Superintendent of the Water Division has been J. S. Nichols. The Water Division operates as part of the Department of Public Utilities, of which initially T. M. Hohl was Manager and more recently, during the major part of the construction, R. D. Nixon was the latter's successor.

The engineering aspects of the development of the project of additional water supply and the construction of Montgomery dam have throughout been in charge of Black & Veatch, Consulting Engineers of Kansas City, Missouri, with N. T. Veatch, M. ASCE, in general supervision and G. R. Scott, M. ASCE, in charge of the project. Throughout the performance of the contract for the construction of the rock fill, 1954-56, Martin Hemker served as Resident Engineer for Black & Veatch; for the construction of the asphaltic concrete deck in 1957, that position was occupied by William Gus Koulas, A.M. ASCE.

On exploration for dam and reservoir site, design and preparation of specifications for rock fill, grouting and asphaltic concrete deck and construction of those elements of the work, the office of F. W. Scheidenhelm, M. ASCE, including the co-authors of this paper, served as consultant to Black & Veatch.

During 1957, in connection with the construction of the asphaltic concrete deck, L. M. Ellsperman contributed information acquired during the Glen Anne dam construction.

The Bituminous Laboratory of the Bureau of Reclamation, located at the Federal Center, Denver, Colorado, and at which there was carried out the extensive pre-construction experimentation, is under the general direction of Walter H. Price, M. ASCE, Chief, Engineering Laboratories of the Bureau, and during that experimentation was under divisional direction by Willis T. Moran. The immediate testing throughout was in charge of L. M. Ellsperman, who since 1954 has been head of the Bituminous Laboratory.

For the State of Colorado, the plans of the rock-fill part of the dam were reviewed and approved by M. C. Hinderlider, State Engineer. Review and approval in the case of the asphaltic concrete deck were by his successor, J. E. Whitten.

In connection with the actual construction of the deck, the laboratory of the Colorado Department of Highways, of which E. G. Swanson is Staff Materials Engineer, did the major part of the pertinent testing.

The contractor for the rock-fill dam was Fisher Contracting Company, Phoenix, Arizona, with Ray Spangler in 1955 and Ken Willis in 1956 as job superintendents. Grouting was sub-contracted to Continental Drilling Co., of Los Angeles, California, with Bond H. Jobe as field superintendent.

The contractor for the construction of the asphaltic concrete deck was San Ore Construction Company, of McPherson, Kansas, with Dean Dillon as job superintendent.

18. Significance of Montgomery Type of Dam

In general, where rock suitable for such fill is readily available, as at Montgomery, the combination of rock fill and asphaltic concrete deck not only

makes maximum use of locally available material but does so at a substantial saving in cost as compared with other types of dam offering equal advantages. Also it permits economical dam construction in areas where there is not available material either for a dam or core of the rolled-earth-fill type or for a dam involving Portland cement concrete but there is available rock good enough for a rock-fill dam.

In general, as compared with decking of steel or Portland cement concrete for a rock-fill dam, the combination in point results in substantial economy. (This was true at Montgomery despite the fact that all materials required for Portland cement concrete excepting the cement itself were available locally.) In addition, there obtain the other advantages pointed out under items 6 and 7 hereinabove.

Although apparently without precedent, at least in the United States, an asphaltic concrete deck of the Montgomery type is believed to have economic possibilities also for situations where material suitable for rock fill is not available but there is available permeable material, such as gravel, disintegrated rock or earth, not adaptable to a conventional rolled fill. Such construction would obviate the need for watertightness of the fill. There would be required compaction only for the purpose of reducing, if not minimizing, settlement. However, such a form of dam would involve important design and construction considerations not involved in a rock-fill dam. As stated in item 5, a modified earth-fill type of construction was considered for the Montgomery site but, without study for detailed design, was found not to indicate sufficiently greater economy as compared with the constructed type of rock fill with asphaltic concrete deck.

The authors have found that, in the case of a hydroelectric development scheduled for construction beginning in 1958 and involving a dam twice as high as that at Montgomery, the project, when appraised on the basis of sound economic criteria, has become feasible by reason of the use of a dam of the rock-fill type (with asphaltic concrete deck), whereas with any other type of dam it would not be feasible.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1449 is identified as 1449 (HY 6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1957.

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FEBRUARY: 1162(HY1), 1163(HY1), 1164(HY1), 1165(HY1), 1166(HY1), 1167(HY1), 1168(SA1), 1169(SA1), 1170(SA1), 1171(SA1), 1172(SA1), 1173(SA1), 1174(SA1), 1175(SA1), 1176(SA1), 1177(HY1)[c], 1178(SA1), 1179(SA1), 1180(SA1), 1181(SA1), 1182(PO1), 1183(PO1), 1184(PO1), 1185(PO1)[c].

MARCH: 1186(ST2), 1187(ST2), 1188(ST2), 1189(ST2), 1190(ST2), 1191(ST2), 1192(ST2)c, 1193 (PL1), 1194(PL1), 1195(PL1).

APRIL: 1196(EM2), 1197(HY2), 1198(HY2), 1199(HY2), 1200(HY2), 1201(HY2), 1202(HY2), 1203 (SA2), 1204(SM2), 1205(SM2), 1206(SM2), 1207(SM2), 1208(WW1), 1209(WW1), 1210(WW1), 1211(WW1), 1212(EM2), 1213(EM2), 1214(EM2), 1215(PO2), 1216(PO2), 1217(PO2), 1218 (SA2), 1219(SA2), 1220(SA2), 1221(SA2), 1222(SA2), 1223(SA2), 1224(SA2), 1225(PO)[c], 1226 (WW1)[c], 1227(SA2)[c], 1228(SM2)[c], 1229(EM2)[c], 1230(HY2)[c].

MAY: 1231(ST3), 1232(ST3), 1233(ST3), 1234(ST3), 1235(IR1), 1236(IR1), 1237(WW2), 1238 (WW2), 1239(WW2), 1240(WW2), 1241(WW2), 1242(WW2), 1243(WW2), 1244(HW2), 1245(HW2), 1246(HW2), 1247(HW2), 1248(WW2), 1249(HW2), 1250(HW2), 1251(WW2), 1252(WW2), 1253 (IR1), 1254(ST3), 1255(ST3), 1256(HW2), 1257(IR1)[c], 1258(HW2)[c], 1259(ST3)[c].

JUNE: 1260(HY3), 1261(HY3), 1262(HY3), 1263(HY3), 1264(HY3), 1265(HY3), 1266(HY3), 1267 (PO3), 1268(PO3), 1269(SA3), 1270(SA3), 1271(SA3), 1272(SA3), 1273(SA3), 1274(SA3), 1275 (SA3), 1276(SA3), 1277(HY3), 1278(HY3), 1279(PL2), 1280(PL2), 1281(PL2), 1282(SA3), 1283 (HY3)[c], 1284(PO3), 1285(PO3), 1286(PO3), 1287(PO3)[c], 1288(SA3)[c].

JULY: 1289(SM3), 1290(EM3), 1291(EM3), 1292(EM3), 1293(EM3), 1294(HW3), 1295(HW3), 1296 (HW3), 1297(HW3), 1298(HW3), 1299(SM3), 1300(SM3), 1301(SM3), 1302(ST4), 1303(ST4), 1304 (ST4), 1305(SU1), 1306(SU1), 1307(SU1), 1308(ST4), 1309(SM3), 1310(SU1)[c], 1311(EM3)[c], 1312 (ST4), 1313(ST4), 1314(ST4), 1315(ST4), 1316(ST4), 1317(ST4), 1318(ST4), 1319(SM3)[c], 1320 (ST4), 1321(ST4), 1322(EM3), 1323(AT1), 1324(AT1), 1325(AT1), 1326(AT1), 1327(AT1), 1328 (AT1)[c], 1329(ST4)[c].

AUGUST: 1330(HY4), 1331(HY4), 1332(HY4), 1333(SA4), 1334(SA4), 1335(SA4), 1336(SA4), 1337 (SA4), 1338(SA4), 1339(CO1), 1340(CO1), 1341(CO1), 1342(CO1), 1343(CO1), 1344(PO4), 1345 (HY4), 1346(PO4)[c], 1347(BD1), 1348(HY4)[c], 1349(SA4)[c], 1350(PO4), 1351(PO4).

SEPTEMBER: 1352(IR2), 1353(ST5), 1354(ST5), 1355(ST5), 1356(ST5), 1357(ST5), 1358(ST5), 1359(IR2), 1360(IR2), 1361(ST5), 1362(IR2), 1363(IR2), 1364(IR2), 1365(WW3), 1366(WW3), 1367(WW3), 1368(WW3), 1369(WW3), 1370(WW3), 1371(HW4), 1372(HW4), 1373(HW4), 1374 (HW4), 1375(PL3), 1376(PL3), 1377(IR2)[c], 1378(HW4)[c], 1379(IR2), 1380(HW4), 1381(WW3)[c], 1382(ST5)[c], 1383(PL3)[c], 1384(IR2), 1385(HW4), 1386(HW4).

OCTOBER: 1387(CP2), 1388(CP2), 1389(EM4), 1390(EM4), 1391(HY5), 1392(HY5), 1393(HY5), 1394(HY5), 1395(HY5), 1396(PO5), 1397(PO5), 1398(PO5), 1399(EM4), 1400(SA5), 1401(HY5), 1402(HY5), 1403(HY5), 1404(HY5), 1405(HY5), 1406(HY5), 1407(SA5), 1408(SA5), 1409(SA5), 1410(SA5), 1411(SA5), 1412(EM4), 1413(EM4), 1414(PO5), 1415(EM4)[c], 1416(PO5)[c], 1417 (HY5)[c], 1418(EM4), 1419(PO5), 1420(PO5), 1421(PO5), 1422(SA5)[c], 1423(SA5), 1424(EM4), 1425(CP2).

NOVEMBER: 1426(SM4), 1427(SM4), 1428(SM4), 1429(SM4), 1430(SM4)[c], 1431(ST6), 1432(ST6), 1433(ST6), 1434(ST6), 1435(ST6), 1436(ST6), 1437(ST6), 1438(SM4), 1439(SM4), 1440(ST6), 1441(ST6), 1442(ST6)[c], 1443(SU2), 1444(SU2), 1445(SU2), 1446(SU2), 1447(SU2), 1448(SU2)[c].

DECEMBER: 1449(HY6), 1450(HY6), 1451(HY6), 1452(HY6), 1453(HY6), 1454(HY6), 1455(HY6), 1456(HY6)[c], 1457(PO6), 1458(PO6), 1459(PO6), 1460(PO6)[c], 1461(SA6), 1462(SA6), 1463(SA6), 1464(SA6), 1465(SA6), 1466(SA6)[c], 1467(AT2), 1468(AT2), 1469(AT2), 1470(AT2), 1471(AT2), 1472(AT2), 1473(AT2), 1474(AT2), 1475(AT2), 1476(AT2), 1477(AT2), 1478(AT2), 1479(AT2), 1480(AT2), 1481(AT2), 1482(AT2), 1483(AT2), 1484(AT2), 1485(AT2)[c], 1486(BD2), 1487(BD2), 1488(PO6), 1489(PO6), 1490(BD2), 1491(BD2), 1492(HY6), 1493(BD2).

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JANUARY: 1494(EM1), 1495(EM1), 1496(EM1), 1497(IR1), 1498(IR1), 1499(IR1), 1500(IR1), 1501(IR1), 1502(IR1), 1503(IR1), 1504(IR1), 1505(IR1), 1506(IR1), 1507(IR1), 1508(ST1), 1509(ST1), 1510(ST1), 1511(ST1), 1512(ST1), 1513(WW1), 1514(WW1), 1515(WW1), 1516(WW1), 1517(WW1), 1518(WW1), 1519(ST1), 1520(EM1)[c], 1521(IR1)[c], 1522(ST1)[c], 1523(WW1)[c], 1524(HW1), 1525(HW1), 1526(HW1)[c], 1527(HW1).

FEBRUARY: 1528(HY1), 1529(PO1), 1530(HY1), 1531(HY1), 1532(HY1), 1533(SA1), 1534(SA1), 1535(SM1), 1536(SM1), 1537(SM1), 1538(PO1)[c], 1539(SA1), 1540(SA1), 1541(SA1), 1542(SA1), 1543(SA1), 1544(SM1), 1545(SM1), 1546(SM1), 1547(SM1), 1548(SM1), 1549(SM1), 1550(SM1), 1551(SM1), 1552(SM1), 1553(PO1), 1554(PO1), 1555(PO1), 1556(PO1), 1557(SA1)[c], 1558(HY1)[c], 1559(SM1)[c].

c. Discussion of several papers, grouped by divisions.

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